The Behaviour of Salinity and Density Stratified Flows in the Wimmera River, Australia

by

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Addendum and Errata for Thesis of A.W. Western (1994)

ADDENDUM

Note: Paragraph numbers refer to paragraphs which form part of the body of the text (headings excluded) and commence with 1 for the first paragraph or part thereof at the top of a page.

Page	Paragraph	Line	Description
105	4	4	"residual, r," to replace "residual"
183			Dispersion coefficients quoted have units of m^2/s
184	Figure 6.17		Dispersion coefficients quoted have units of m^2/s
197	1	1	insert "Of particular interest in water quality modelling is " at the start of the first sentence
206	5	5	insert "where K is the molecular diffusivity, " before "becomes"

ERRATA

Note: Paragraph numbers refer to paragraphs which form part of the body of the text (headings excluded) and commence with 1 for the first paragraph or part thereof at the top of a page.

Page	Paragraph	Line	Description
xv	Appendix 1		"Solution" to replace "Soultion"
65	4	1	"canvasses" to replace "canvases"
74	3	7	"salinity" to replace "salinty"
109	Equation 5.2		"Z" to replace "D"
109	4	2	"Z/Z_{bf}" to replace "A/Z_{bf}"
113	Figure 5.9		"bank-full channel depth" to replace "channel depth"
172	3	2	"ω" to replace "V"
178	Figure 6.13		"1/1/92" to replace "1/1/93" four times
209	Equations 7.3b-7.3d		"Ri_b" to replace "Ri"
252	1	2	"ellipsoid" to replace "ellipse"
252	1	2	"π" to replace "Π"
265	Figure 8.44		"at points" to replace "a tpoints"
309	Figure 9.25		"release" to replace "realease"
338	Figure 10.1		"ω" to replace "W" twice

Declaration

This thesis contains no material that has been accepted for the award of any other degree or diploma at any university or institution and, to the best of the author's knowledge and belief, contains no material previously written or published by another person except where due reference is made in the text. The length of this thesis is less than 100 000 words excluding tables, figures, references and appendices.

Andrew Western, November 1994.

Abstract

A quantitative understanding of the behaviour of salinity and density stratification in the Wimmera River is developed using a combination of field, laboratory and numerical modelling techniques. The Wimmera River, which is located in north-western Victoria, Australia, is a saline stream with a seasonal and highly variable flow regime. Large salt fluxes enter the Wimmera River as a result of surface water inflows from the upper catchment and groundwater inflows in the upper and lower reaches of the stream. During periods of low and zero flow, a series of long deep pools exist along the river, particularly downstream of Horsham. Inflows of saline groundwater accumulate in scour depressions within these large pools and a series of density stratified or saline pools results. Small flows of saline and fresh water down the river can also lead to density stratification. Larger flow events lead to destruction of the density stratification.

A model of flow and salinity in a 200 km reach of the Wimmera River is developed using the MIKE 11 model (DHI, 1992a; 1992b). MIKE 11 solves the St Venant equations for gradually varied, unsteady flow and the Advection-Dispersion equation for solute transport. A time-series data base of discharges and salinities for all surface water and groundwater inflows to the river is developed. This was an important step in the model development due to the existence of a significant number of ungauged tributaries and the importance of groundwater as a source of salt.

Stream channels are specified in MIKE 11 by defining a channel network and specifying a series of cross-sections along each channel. The channel morphology of the Wimmera River is studied and a methodology for characterising channel variability is developed. It is shown that the Wimmera River channel can be divided into two statistically different channel types which are characterised by a typical length-scale of several kilometres. Using the above analysis as a basis, a stochastic model of stream channel cross-sections is developed for the Wimmera River and used to infill the existing cross-sectional data. The hydraulic implications of along-channel cross-sectional variation are investigated numerically.

A one-dimensional model of the Wimmera River is calibrated and tested. This model is applicable to in-bank flows and their associated salinities. The model adequately simulates the routing of water and salt down the Wimmera River. Variations in salinity associated with flow events and the seasonal variation of salinity are reproduced.

Field and laboratory investigations of density stratified pools are described. Density stratified pools form as a result of saline groundwater inflows when the stream discharge is less than 200 - 300 Ml/d. The rate at which the stratification develops is quantified for four field sites. Saline water is mixed from density stratified pools during flow events. The mechanism responsible for most of the mixing involves a thin outflow of saline water up the downstream slope of the scour depression. Turbulent entrainment is also responsible for some mixing. During the autumn, convection associated with surface cooling can also mix some density stratified pools.

A model of individual density stratified pools, known as Salipool, is developed and tested. Salipool is applied to four density stratified pools in the Wimmera River. A generalised calibration of the mixing relationship incorporated in Salipool is suggested. This generalisation is based on bend sharpness. It is hypothesised that bends have a significant impact on mixing of density stratified pools due to their effect on the vertical velocity profile and the direction of near-bed currents. Salipool is used as a basis for modifying MIKE 11 to incorporate the effect of density stratification.

Acknowledgements

This project has involved research into the behaviour of salinity and density stratification in the Wimmera River. It was motivated by the need for an improved understanding of this behaviour so that the problems associated with salinity and density stratification could be better managed. Funding for this project was provided by the University of Melbourne through an Australian Postgraduate Research Award and by the Rural Water Corporation of Victoria. The research was carried out within the Centre for Environmental Applied Hydrology.

An important feature of the project was the cooperation with the Rural Water Corporation of Victoria (RWC). While the involvement of the RWC placed an emphasis on the practical outcomes of the research, personnel at the RWC recognised that a fundamental understanding of the stream system was required to achieve these outcomes. As a result it was possible to pursue issues that arose during the research with a significant amount of freedom. That freedom is greatfully acknowledged. Assistance from the following RWC personnel is also greatfully acknowledged.

- Mr Geoff Earl instigated the project and organised the funding provided by the RWC.
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- Mr John Martin checked digitised data from the Wimmera Mallee Stock and Domestic Water Supply System (WMSDS) against the original paper records and made the necessary corrections.
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The general direction of the research was set in consultation with a steering committee consisting of my supervisors, Mr David Hooke, Mr Erwin Weinmann and myself.

Finally thanks to my parents for their support over the years and for believing in the value of a good education and to my friends for their support and for going skiing and bush-walking with me. Many thanks to my partner, Judy Dunai, without whose emotional support this project and especially the preparation of this thesis would have been much more difficult.

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A, A_1, A_2	catchment area
A	cross-sectional area
A	main effect of σ_a
A_b	boundary area
A_{bf}	bank-full cross-sectional area
A_c	solute rating curve coefficient
A_D	cross-sectional area at downstream limit of stratification
A_f	inundated area
A_p	plan area
A_s	flow area at a hydraulic structure (weir)
AB	interaction between σ_a and σ_b
AR	interaction between σ_a and σ_r
ABR	interaction between σ_a, σ_b and σ_r
a	coefficient in channel width depth relationship
a^*	bank-full width to depth ratio
a	solute rating curve coefficient
B	main effect of σ_b
B	water surface width
B_b	instantaneous base flow index
B_c	constant in solute rating curve
B_i	interface width
B_{st}	storage width
BR	interaction between σ_b and σ_r
b	solute rating curve index
b	cross-section shape factor
b'	perturbation in channel width
C	salinity
C, C_1, C_2	concentration
C_E	entrainment coefficient
C_{eff}	coefficient of efficiency
C_α, C_o	fixed concentration
C_δ	concentration difference
C_f, C_t, C_d	parameterisation coefficients for turbulent kinetic energy relationship
C_F	flushing coefficent
Ch	river chainage
C_{MVUE}	concetration estimated by minimum variance unbiased estimator

Cr	Courant number
C_s	lateral inflow solute concentration
c	kinematic wave celerity
c'	modified kinematic wave celerity
cv	coefficient of variation
D	hydraulic mean depth
D	dispersion coefficient
d	depth
$d\theta$	incremental channel bend angle
$E = U_c/U$	entrainment parameter
EC	electrical conductivity measured in µmhos/cm of µS/cm at a temperature of 25°C
E_*	entrainment parameter based on shear velocity
$F = Q_f/U_1LB$	flushing parameter
F	F statistic
$Fr = U/\sqrt{gD}$	Froude number
f	friction factor
f_b	boundary friction factor
g	gravitational acceleration
$g' = (\Delta\rho/\rho)g$	buoyancy
H	water surface elevation
H'	perturbation of mean channel bed elevation from mean channel grade line
ΔH	head loss
h	mixed layer depth
h	elevation
h_1	upper layer depth
h_2	lower layer depth
I	inflow
K	storage parameter in Muskingum equation
K	conveyance
K_{sat}	saturated hydraulic conductivity
K_D	molecular diffusivity
k	storage parameter
k	roughness height
k_D	parameter relating dispersion coefficient to velocity
$L = CV$	solute load
L	interface length
L_b	basin length

L_m	mixing length
l	length scale
$\mathbf{M_r}$	radial angular momentum
$\mathbf{M_\theta}$	streamwise angular momentum
m	degrees of freedom
n	sample size
n	Manning's n, flow resistance parameter
n	index for catchment storage discharge relationship
O	outflow
P	wetted perimeter
Pe	Peclet Number
P_s	proportion of total change in salt flux due to a change in salinity
Q, Q_1, Q_2	discharge
Q_b	baseflow discharge
Q_f	volumetric rate of saline pool flushing
Q_{fD}	volumetric rate of saline pool flushing due to downstream outflow
Q_{gw}	volumetric rate of groundwater inflow to saline layer
Q'_{gw}	volumetric rate of groundwater inflow to a river reach
Q_i	interflow discharge
Q_{mix}	volumetric mixing rate
Q_s	surfaceflow discharge
Q_0	threshold discharge
$Q_{15}, Q_{30}, Q_{45}, Q_{60}$	n day antecedent discharge
Q^*	discharge for which concentration is estimated
q	turbulent kinetic energy
q	lateral inflow
q	flow per unit width
R	main effect of σ_r
R	hydraulic radius
R^2	coefficient of determination
R_b	bend radius
$Re = ul/\nu$	Reynold's Number
Ri	Richardson Number
Ri_b	Bulk Richardson Number
Ri_g	Gradient Richardson number
Ri_*	Shear Richardson Number
Ri_{*b}	Boundary Shear Richardson Number

Ri_{vr}	Richardson Number based on return velocity
Ri_{Ls}	Richardson number based on interface length and g'tanθ
Ri_{Ls}	Richardson number based on interface length and g'
R_l	ratio of maximum to mean interface length
r	surfaceflow to baseflow ratio
r	residual from regression relationship between channel invert level and chainage
r	radius of curvature of a streamline in the horizontal plane
r	correlation coefficient
S	solute storage
S	river stage
S_f	friction slope
S_o	bed slope
s	unbiased estimate of standard deviation
SSM	sum of squared differences from the mean
SSR	sum of squared residuals
T	time interval between concentration samples
T_d	dissipation time scale for an internal seiche
T_{high}	time river discharge exceeded 1000 Ml/d
T_i	period of an internal seiche
TDS	total dissolved solids
TKE	Turbulent Kinetic Energy
T_δ	diffusive interface formation timescale
T_l	interface erosion timescale
T_{250}	time since river discharge exceeded 250 Ml/d
T_{500}	time since river discharge exceeded 500 Ml/d
Δt	time difference, time step
t	time
U	mean velocity
U'	mean velocity in stratified part of the channel
U_c	Critical velocity
U_e	entrainment velocity
U_{max}	maximum velocity
ΔU	velocity difference across density interface
U_1	mean upper layer velocity
U_{1D}	mean upper layer velocity at downstream limit of stratification
u	velocity scale
u*	shear velocity

V	(water) storage volume
V_b	basin volume
V_{gw}	equivalent groundwater volume
V_n	scour depression volume
V_o	cease to flow storage volume
V_p	variance of regression predictions
V_s	volume of saline layer
v	streamwise velocity
W	channel width
$W^*=W/Z_{bf}$	non-dimensional width
w'	vertical turbulent velocity fluctuation
X	weighting parameter in Muskingum equation
Δx	a finite distance, a space step
y	water depth
Z	height above channel invert
Z_{bf}	bank-full channel depth
$Z^* = Z/Z_{bf}$	non-dimensional depth
z'	purturbation in channel bed elevation
α	momentum distribution coefficient
α	baseflow filter constant
δ	interfacial thickness
δ_b	boundary layer thickness
δ_u	velocity shear layer thickness
ε	density ratio
ε_t	coefficient of transverse turbulent diffusion
γ	value of semi-variogram function
ρ	density
ρ'	turbulent density fluctuation
ρ	density of water
ρ_o	reference density
$\Delta\rho$	interfacial density jump
φ	proportion of groundwater assigned to saline layer
σ	turbulence integral velocity scale
σ	standard deviation
σ_z	vertical integral velocity scale at a density interface
τ_o	mean boundary shear stress
μ	mean
ν	kinematic viscosity
θ	bed slope

ω	storage width calibration parameter
ζ_{in}	inflow loss coefficient for broad-crested weir
ζ_{out}	outflow loss coefficient for broad-crested weir

List of Abbreviations

DCE	Department of Conservation and Environment, Victoria
KFRS	Kaiela Fisheries Research Station
RWC	Rural Water Corporation of Victoria
WCCG	Wimmera Catchment Coordinating Group
WMSDS	Wimmera Mallee Stock and Domestic Water Supply System

CHAPTER 1 - INTRODUCTION

1.1 INTRODUCTION.

In 1853 a Victorian grazier reportedly wrote to Governor La Trobe lamenting: "now the soil is getting trodden hard with stock, springs of salt water are bursting out in every hollow of watercourse, and as it trickles down the water course in summer, the strong tussocky grasses die before it with all others" (Peck, 1993). Today soil and water salinisation continues in irrigated and dry-land (non-irrigated) landscapes in south eastern and south western Australia (Loh, 1988; Allison et al., 1990; Macumber, 1991; Ghassemi et al., 1991; Peck 1993). This is primarily the result of a major change in the hydrologic balance caused by changes to vegetative systems and the commencement of irrigation. Replacement of native vegetation with shallow rooted crops and pastures since European colonisation has substantially increased groundwater recharge leading to rising water tables and consequent salinisation (Ghassemi et al., 1991; Macumber, 1991).

Both land and stream salinisation have been recognised as significant problems in parts of Australia since early this century (Loh, 1988). While the visual impacts of land salinisation are often spectacular and the associated production losses are easily imagined; the impact of salinisation on streams may be an even greater problem in some areas, particularly in parts of Western Australia and western Victoria (Peck, 1993). McGuckin et al. (1991) note that salt affected rivers and streams occur throughout half of Victoria, notably on the lower lying plains in northern and western Victoria. Stream salinisation is important from both the water supply and environmental perspectives.

In 1989 a major report on the aquatic environment in the Wimmera River, which is located in north western Victoria, was released (Anderson and Morison, 1989a-g). High stream salinities and density stratification associated with the inflow of saline groundwater were identified as major environmental problems. The term saline pool was used to describe density stratified pools. Saline pools consist of an upper layer of fresh water and a lower layer of saline water. The saline layer is typically contained within a scour hole in the stream. The saline water, which is usually anoxic and acidic, is unsuitable for habitation by fish and other aerobic organisms (Anderson and Morison, 1989c; 1989g). Similar saline pools exist in other Victorian streams (McGuckin et al., 1991). While Anderson and Morison (1989c; 1989g) studied the seasonal behaviour of saline pools in some detail, the processes controlling the behaviour of saline pools were only considered qualitatively.

High salinities in the Wimmera Mallee Stock and Domestic Water Supply System (WMSDS), which are partly related to the salinity of the Wimmera River, are of concern to the Rural Water Corporation of Victoria (RWC) and consumers relying on water from the WMSDS. There is increasing community concern about water quality and the impact of the WMSDS on the aquatic ecosystem in the Wimmera River (WCCG, 1991; WCCG, 1992). Given these problems, there is a need for a quantitative understanding of the transport of salt through the Wimmera River system. This study was performed with the aim of developing that understanding, particularly in relation to saline pools.

While this study has focused on the Wimmera River many of the results, particularly those relating to saline pools, are applicable to other saline streams. In this regard it is noteworthy that saline pools exist in many saline streams. Furthermore, a critical issue in many saline areas is the disposal of saline water (Peck, 1993). Proper assessment of proposals to dispose of saline water to streams such as the Goulburn River in Victoria will require an understanding of the movement of saline water in streams (McGuckin, 1991).

1.2 METHODOLOGICAL APPROACH.

This study aimed to develop a quantitative understanding of the processes controlling the behaviour of salinity in the Wimmera River and to make that understanding available to river managers and any other interested parties. For the results of this project to be practically useful, it was necessary to be able to predict the impact of changed flow management on stream salinities. Such predictions were required so that future flow management options, such as the provision of environmental flows, could be properly assessed. Furthermore those predictions needed to be made for locations up to 150 km away from the point where the change was made. Development of a sound understanding of the stream behaviour was critical to making proper predictions and required a theoretical framework in which interactions between various parts of the river system could be explored and hypotheses relating to the behaviour of the river system could be developed and tested. Use of a model was seen as essential to satisfying the above requirements.

Before starting to develop a model it was to necessary to decide which features of the hydrology of the river needed to be modelled and to which parts of the river system the model should apply. It was known *a priori* that stream salinity depended on the flow and that the flow regime was characterised by long periods of low flow interspersed with flow events resulting from runoff during storms in the catchment. Therefore the model needed to be capable of simulating fluctuations in flow and salinity at both the event time-scale (several hours) and at long time-scales (more than a year).

Given that the interest was in the movement of salt within the river and between river and the WMSDS, the model was limited to a consideration of the river and locations where the river and the WMSDS interacted. Since river salinities are generally small during floods and thus relatively unimportant from an in-stream water quality perspective, the modelling focused on in-bank flows. Saline pools needed to be incorporated in the model so that their effect on stream salinity could be investigated. The upstream and downstream limits of the model were placed at gauging stations so that hydrologic data were available at the model boundaries. Approximately 200 km of the Wimmera River, more than half its length, were modelled.

It should be emphasised that this project was not simply a modelling exercise. Much of the understanding of the Wimmera River developed during this study resulted from complementary investigations. For instance field investigations were crucial in developing an understanding of the behaviour of saline pools. The model was used subsequently to integrate the results from the different investigations.

Since there was only limited knowledge of the behaviour of saline pools, a program of field investigations into the processes determining their behaviour was developed. These field investigations were performed in parallel with the model development. From initial field investigations it became clear that a series of laboratory experiments would be useful for clarifying some aspects of the flushing of saline pools. These experiments were conducted by Nolan (1994). The understanding of the behaviour of saline pools developed primarily through the field and laboratory investigations combined with numerical modelling of specific saline pools.

While the river modelling did not consider the catchment of the Wimmera River explicitly, flows of water and salt from the catchment have a significant impact on the river. Interaction between the groundwater and the river is also important. A significant effort was required to synthesise existing information relating to inflows of surface and groundwater and to estimate those flows where required.

An understanding of the characteristics of the Wimmera River channel was also required and this was developed by analysing existing cross-sectional information.

1.2.1 UNITS OF FLOW AND SALINITY.

The units used for flow and salinity in this thesis are not standard international units. Rather the units are those that are commonly used in local practice. This is in keeping with the practical nature of this research. Discharge is usually provided in Megalitres per day (Ml/d) and may be easily converted to m^3/s by dividing by 86.4. Salinity is usually determined by measuring the electrical conductivity of the water and the measurements are provided in the units of μmhos/cm or μs/cm at the standard temperature of 25° C. These units are commonly referred to as EC units and this convention is followed in this thesis. The following equation can be used to standardise salinity measurements performed at temperatures other than 25° C (Peck, 1993).

$$EC_{25}/EC_T = 1.019 - 0.0208(T - 25) \tag{1.1}$$

Salinities quoted in EC units can be convert to parts per million (TDS) by multiplying by 0.6 (D. Hooke, RWC, Per. Com.).

1.3 THESIS LAYOUT.

The structure of this thesis reflects the logical progression of the research. The Wimmera River and its catchment are described in detail in Chapter 2. In the Wimmera River catchment the climate varies from sub-humid to semi-arid and flows are highly variable and saline. Inflows of saline groundwater to the Wimmera River are large and these lead to formation of saline pools which have important environmental consequences. The available data for this study are reviewed at the end of Chapter 2.

A discussion of the theory of one-dimensional flow and solute transport in natural streams is provided in Chapter 3. The St Venant equations can be used to describe the routing of flood waves down a stream. The routing process is dominated by storage of water in and adjacent to the stream channel. Solute transport can be described with the Advection-Dispersion equation. The process of longitudinal mixing is discussed in some detail. Modelling of streams is discussed and the reasons for selecting the MIKE 11 model (DHI, 1992a; 1992b) for modelling the Wimmera River are given. Relevant aspects of MIKE 11 are discussed.

Inflows of surface and groundwater occur at many points along the section of the Wimmera River that was modelled. Chapter 4 describes how the discharges and salinities were estimated for each of these inflows, many of which were ungauged. Discharge from ungauged tributaries was estimated by scaling flows from a nearby stream. The ratio used for this scaling was obtained from a relationship between mean annual flow and catchment area developed for streams in the region.

The salinity in the Wimmera River at the upstream extremity of the model and in ungauged tributaries also required estimation. Solute rating curves, which are relationships between flow and concentration, were used to make these estimations. For the Wimmera River the solute rating curve was developed using regression techniques. At other sites an understanding of the typical variation of salinity with flow for streams in the region was combined with the typical salt production per unit area to estimate appropriate solute rating curves. The interaction between the Wimmera River and the groundwater was studied in detail for the upper river. Estimates of groundwater inflows obtained from another study were used to specify groundwater inflow to the lower Wimmera River.

In Chapter 5 the geometry of the Wimmera River channel is considered in detail. It is shown that there are two distinct categories of channel: one where the river channel is anabranching and one where a single channel exists. The anabranching areas are referred to as wetlands. Variation between the two types of channel occur at a scale of several kilometres. This is considerably greater than the meander length scale. A detailed statistical analysis of existing cross-sections from the Wimmera River is used as the basis for a stochastic method for infilling cross-section data.

The hydraulic implications of channel variability are examined using a series of numerical simulations. It is shown that predictions of stage, depth and residence time are sensitive to channel variability at low flows and that channel specification is critical when local predictions are of interest. Furthermore it is shown that flow friction is more important than channel variability in determining flow routing behaviour.

A one-dimensional model of the Wimmera River is presented in Chapter 6. The model structure and idealisations of the physical system are described. Stage-discharge relationships are used to calibrate the flow resistance parameter. After calibrating the flow resistance parameter, the model predicted flood wave celerities which were significantly too large. Although a number of possible explanations exist, it is considered that lack of knowledge of additional storage in relict channels and low lying areas in wetlands are the likely causes of this error. Additional storage was added to the model to correct the predictions of flood wave celerity.

The model predicts flows well in the upper part of the river; however further downstream greater errors are present. The fact that the total water volume in many events is incorrect suggests many of that the simulation errors are related to errors in estimated tributary discharges. During high flows stream salinities are generally well predicted and are dominated by surface water inflows. However significant errors occur in the lower part of the river during low flow periods. Groundwater inflows have a major influence on salinity in the Wimmera River downstream of Horsham during low flow periods.

Following Chapter 6 the theme of the thesis changes to a consideration of saline pools. The research into saline pools developed in parallel with the work described in the earlier parts of the thesis. In Chapter 7 a review of the stratified flow literature relevant to saline pools is provided. The entrainment process and the dependence of vertical mixing on different flow characteristics is examined. Mixing due to convection is considered briefly. A series of experiments examining the flushing of saline water from a square cavity is described and limitations of these experiments in relation to the flushing of saline pools are identified.

A detailed analysis of field and laboratory data relating to the formation and mixing of saline pools is performed in Chapter 8. The seasonal behaviour of saline pools is primarily related to the seasonal nature of the flow regime in the Wimmera River. Saline pools are present during low flow periods and are mixed by flows of approximately 1 000 Ml/d depending of the specific site and characteristics of the hydrograph. Saline pools only exist in some scour depressions in the Wimmera River. This is probably due to the inflow of groundwater being localised. Mixing of saline pools by fresh overflows is examined in detail and related to a non-dimensional Richardson number. It is hypothesised that secondary flows in bends have an important influence on the mixing of saline pools. It was found that convection associated with surface cooling could also mix some saline pools. Finally the rate at which saline pools form is examined.

A model of individual saline pools is developed in Chapter 9. This work builds on the analyses in the previous chapter. The model, called Salipool, is applied to four saline pools in the Wimmera River and is able to predict saline pool behaviour adequately. A generalised calibration for the mixing component of the model based on the bend geometry is suggested. Field and laboratory data on mixing in saline pools support the hypothesis that the characteristics of flow around bends are important in mixing of saline pools; however further research is required to confirm this.

The final part of this chapter draws the one-dimensional modelling and the research into saline pools together when a description of saline pools is incorporated in MIKE 11. By comparing simulations with and without saline pools in the river model, the effect of saline pools on salt transport in the Wimmera River is explored. It is shown that saline pools lead to lower surface salinities during low flow periods primarily because they store saline groundwater. Simulations suggest that mixing of saline pools can lead to temporary increases in stream salinity during flow events; however field data do not support this hypothesis.

Several important issues related to, influxes of water and salt to the Wimmera River, stream channel morphology, wetland hydraulics and hydrology and the behaviour of saline pools have arisen from this research. Opportunities for further research and investigation of these issues are discussed in Chapter 10. A detailed discussion of several important issues relating to the use of sophisticated stream models is also provided in Chapter 10. Chapter 11 presents the conclusions of this research.

CHAPTER 2 - THE WIMMERA RIVER BASIN.

This chapter provides background information on the Wimmera River and its catchment. After a brief background, the catchment and its streams are described. The physiography, climate, hydrogeology, water supply system and the flow and salinity regimes are discussed briefly. A previous study of the Wimmera River by Anderson and Morison (1989a-g) is reviewed and the data used in the remainder of this study are discussed.

2.1 PREAMBLE.

The Wimmera River catchment is located in north-western Victoria, Australia (Figure 2.1). Since European colonisation 86% of the total catchment area has been cleared (WCCG, 1991). In the cleared portion of the catchment the dominant land-use is agriculture, especially cereal cropping and sheep grazing (LCC, 1985). The total population in the area is between 30 000 and 40 000. Horsham, Dimboola, Nhill, Warracknabeal and Stawell are the major urban centres within the predominantly rural catchment. Horsham is the largest urban centre in the region and has a population of 12 000 (LCC, 1985).

2.2 CATCHMENT CHARACTERISTICS.

2.2.1 PHYSIOGRAPHY.

From the perspective of the surface water system, the Wimmera River Catchment is a closed catchment. The headwaters of the Wimmera River and its major tributaries lie in the Pyrenees and Grampians Ranges (Figure 2.1). These ranges form the south-western limit of the Great Dividing Range. In the Pyrenees elevations reach 800 m above sea level and there is extensive undulating country and plains. The Grampians are higher (1100 m) and more rugged than the Pyrenees.

The Wimmera River itself flows generally north-west from its origin in the Pyrenees Ranges to Horsham. Downstream of Horsham the river turns and flows north to Lake Hindmarsh which marks the beginning of a system of terminal lakes. All major tributaries, with the exception of Wattle Creek enter the Wimmera River from the south. These tributaries originate in the ranges. No significant tributaries enter the Wimmera River after it has started to flow northwards and the catchment is poorly defined along this reach.

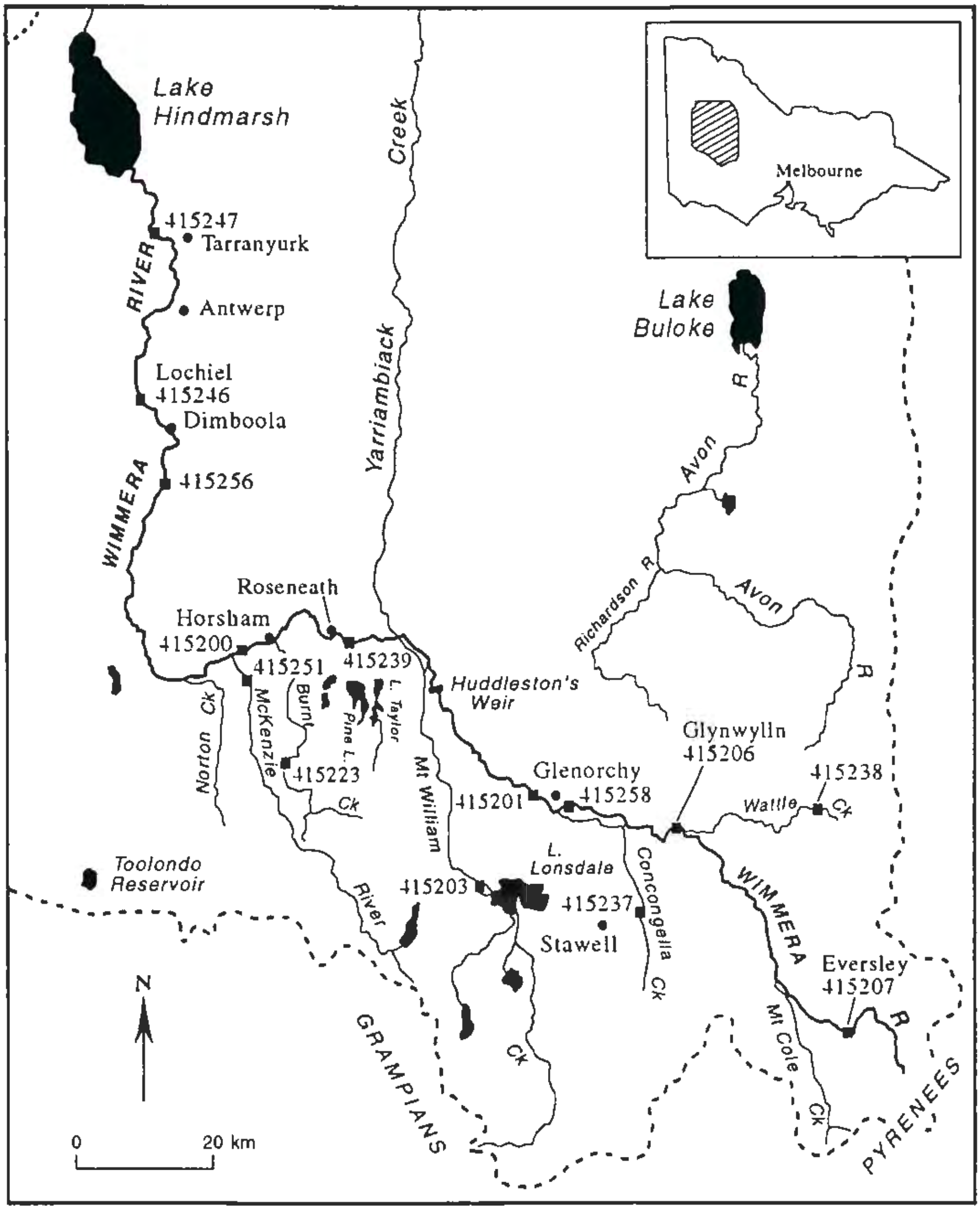


Figure 2.1: The Wimmera River Catchment.

An interesting feature of the Wimmera River system is the existence of several effluent streams of which the Yarriambiack Creek is the most significant (Figure 2.1). Originally water only flowed from the Wimmera River into the Yarriambiack Creek during floods; however today a weir diverts a proportion of low flows in the Wimmera River into Yarriambiack Creek. Swedes and Dunmunkle Creeks (not shown) are two other small effluent streams which leave the Wimmera River near Glenorchy (Figure 2.1) which only flow during floods.

Downstream of Glenorchy the Wimmera River flows through the Wimmera Plains. The Wimmera Plains lie to the north of the ranges and are flat alluvial plains underlain by Tertiary sedimentary deposits of both marine and non-marine origin (LCC, 1985). Annual surface runoff from the Wimmera Plains is normally less than 25 mm (Barlow, 1987) and in most locations it drains to local depressions rather than to the river system.

2.2.2 CLIMATE.

The climate in the Wimmera River catchment varies from sub-humid in the Grampians and Pyrenees Ranges to semi-arid in the north. Mean daily temperatures vary from 12° C minimum and 30° C maximum in summer to 4° C minimum and 14° C maximum in winter. There is an increase in temperatures from south to north in the Wimmera with the ranges generally being cooler (LCC, 1985).

Rainfall in the Wimmera decreases from south to north. The highest rainfall in the catchment occurs in the Grampians where the mean annual rainfall exceeds 1000 mm. In the higher parts of the Pyrenees the mean annual precipitation is 700 mm. On the Wimmera Plains the mean annual rainfall is generally between 400 mm and 500 mm (Barlow, 1987). Winter rainfall is of low intensity while summer rain tends to result from irregular, high intensity thunderstorms. Annual potential evaporation is high in the Wimmera and greatly exceeds annual rainfall in all areas except the Ranges (LCC 1985,1978).

2.2.3 HYDROGEOLOGY.

Interaction between the Wimmera River and the groundwater is important from the perspective of river salinity. Saline groundwater seepage entering the Wimmera River has been observed over extensive reaches of the river, especially downstream of Horsham (Anderson and Morison, 1989c). The groundwater adjacent to the Wimmera River is typically highly saline and any discharge to the river has the potential to significantly increase river salinities. There is potential for groundwater inflow to the Wimmera River upstream of Glenorchy and downstream of Roseneath (C. McAuley, RWC, Per. Com.). A brief description of the hydrogeology in each of these areas is provided.

Foley (1992) summarises the available hydrogeologic information for the Wimmera River Catchment upstream of Glenorchy. There are two major aquifers present in this part of the catchment. The first consists of coarse gravel and sand deposited above the Palaeozoic basement. This gravel and sand is generally overlain by clay and forms a confined aquifer system. The second is a fractured rock aquifer in the Palaeozoic basement. In the area between Glynwylln and

Glenorchy, which is the upstream limit of the area of interest in this study, the lower alluvial unit is typically an angular quartz gravelly clay interbedded with cemented gravel and course sands and is 20-40 m thick. The clay layer in this area is typically 15 - 25 m thick and consists of interbedded clay, sandy clay and clayey silt. The water table is located within the clay layer. The salinities in the confined aquifer in this area typically increase from 3000 to 7000 mg/l Total Dissolved Solids (TDS) with increasing distance from the river.

The groundwater systems which interact with the Wimmera River downstream of Horsham are part of the regional groundwater system in the Murray Basin. The following description is based on a review of the hydrogeology of the Murray Basin by Evans and Kellett (1989); however it is restricted to the Wimmera region.

The Murray Groundwater Basin which consists of 200-600 m of Cainozoic unconsolidated sediments and sedimentary rocks and can be divided into three groundwater provinces. The lower Wimmera River lies in the south-eastern corner of the Mallee-Limestone Province. Major aquifers present in this area are located in the Renmark Group and Pliocene Sands. The Renmark Group lies below the Pliocene Sands and the two are separated by the Gerra Clay and Winnambool Formation which form an aquitard.

The Pliocene Sands Aquifer, which is represented by the Parilla Sands, is generally unconfined and receives recharge from downward leakage of rainfall (Evans and Kellett, 1989). The Parilla Sands consist mainly of well sorted silt and fine to coarse-grained quartz sand (Lawrence and Abele, 1976). Groundwater salinity in the Parilla sands generally varies between 30 000 mg/L TDS and 40 000 mg/L TDS but can reach 300 000 mg/L TDS under refluxing discharge lakes (Evans and Kellett, 1989). Saline water from the Parilla Sands may enter the Wimmera River either directly or via fluvial sediments (Smart, 1989).

2.2.4 THE WATER SUPPLY SYSTEM.

Flows in the Wimmera River and some of its tributaries are modified by water harvesting and distribution activities associated with the Wimmera Mallee Stock and Domestic Water Supply System (WMSDS). As approximately 48% of the annual yield of the Wimmera River and its tributaries is diverted to the WMSDS (Hooke, 1991), the modification of flows in the Wimmera River is significant. A discussion of relevant features of the WMSDS is provided below.

The WMSDS consists of 12 storages and 12 000 km of earthen channels which supply water to an area of 28 000 km^2 (Barlow, 1987). Two-thirds of the water supplied by the WMSDS is harvested from the Wimmera catchment. This is supplemented by transfers north across the Great Dividing Range from the Glenelg and Wannon River catchments (Hooke, 1991). It is generally accepted that water demands on the WMSDS exceed the firm yield of the system (Barlow, 1987) and that the WMSDS is operating with a low level of supply security compared to other Victorian water supply systems (WCCG, 1991). While there are few opportunities for new storages (WCCG, 1991), losses in the earthen water supply channels are high (70% on average and up to 90% in some areas (Anderson and Morison, 1989a)) and improved efficiency of water distribution (eg. by pipe-lining parts of the water supply system) offers the opportunity of improving the security of the system. The WMSDS is usually operated during the winter to minimise losses from the channels and water is stored in farm dams for later use (Barlow, 1987).

The WMSDS interacts directly with the Wimmera River at two locations: Glenorchy and Huddlestons Weir. Glenorchy is a key location in the WMSDS system. A weir in the river increases the water level creating the Glenorchy Weir Pool and the Main Central Channel crosses the Wimmera River at the Glenorchy Weir Pool (Figure 2.2). Water enters the weir pool from Lake Lonsdale via the Main Central Channel and from the Wimmera River and it is either diverted into the Main Central Channel or passes over the weir. The point where the Main Central Channel enters the weir pool is actually downstream of the point where it leaves. This means that water originating from the Wimmera River tends to be diverted and water from Lake Lonsdale tends to pass down the river. Since the water from Lake Lonsdale is usually less saline than the river water, a reduction in the river salinity often occurs when the WMSDS is operating.

A major diversion from the Wimmera River occurs at Huddlestons Weir (Figure 2.1). Up to 1 600 Ml/d of water can be diverted from the river at this location. The diversion is operated to maximise water harvesting and the entire river flow is often diverted at this point. Water diverted at Huddlestons Weir flows via the Wimmera Inlet Channel to Pine Lake and Lake Taylor. There is currently no formal provision for environmental water allocations and the impact of the WMSDS on the Wimmera River is a matter of considerable public concern (WCCG, 1991). Furthermore there is concern about high levels of salinity in the water supply system which are partly due to high salinities in diversions from the Wimmera River (WCCG, 1991; 1992).

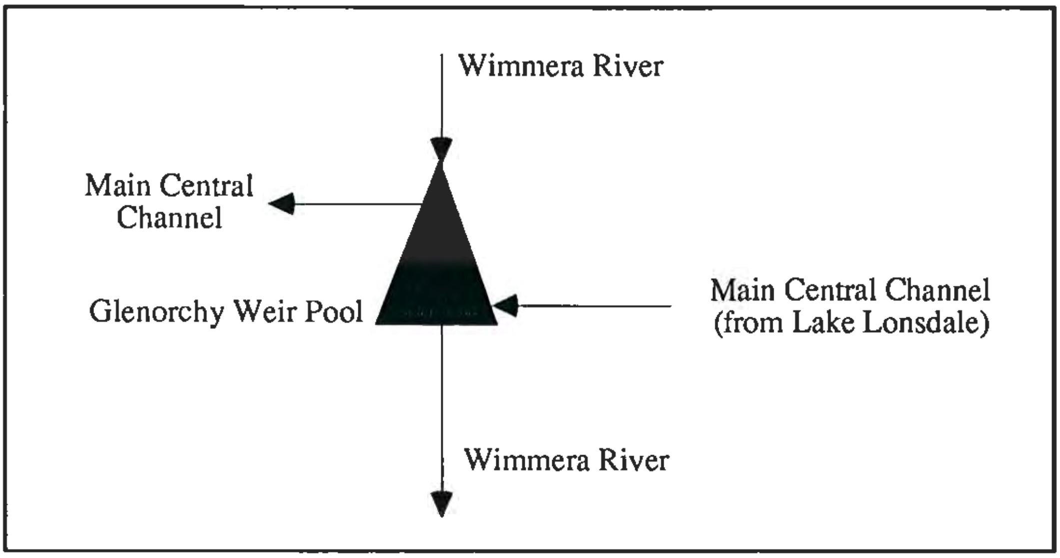


Figure 2.2: Interaction between the Wimmera Mallee Stock and Domestic Water Supply System and the Wimmera River at Glenorchy.

2.3 HYDROLOGY OF THE WIMMERA RIVER.

2.3.1 FLOW REGIME.

The present day flow regime in the Wimmera River is characterised by decreasing runoff depths and increasing variability with distance downstream. Table 2.1 provides mean annual discharges, the coefficient of variability for the mean annual discharge, catchment areas and mean annual runoff depths for four gauging stations along the Wimmera River. In their head-waters the major tributaries and the Wimmera itself are perennial upland streams. Downstream of Huddlestons Weir the Wimmera River is best characterised as an intermittent stream in that it regularly ceases to flow during summer. However, unlike many intermittent streams, a series of large pools remain along the Wimmera River when it ceases to flow. These pools are up to 10 m deep, 100 m wide and 4 km long. Downstream of Lake Hindmarsh, the Wimmera River (now known as Outlet Creek) becomes an episodic stream.

Flows in the Wimmera River are highly seasonal (Figure 2.3). Typically low or zero flows occur from November to June and a series of flow events would be expected from July to October, although this pattern varies significantly from year to year. Extended periods of high flow generally do not occur in the lower Wimmera River except during wet years. Given the seasonal nature of the flow regime, the flow in the lower Wimmera River is typically low.

Gauging Station	Catchment Area (km^2)	Mean Annual Discharge (Ml)	Coefficient of Variation	Mean Annual Runoff (mm)
415207	300	28 300	0.61	94.3
415206	1 357	67 700	0.88	49.9
415201	1 953	92 700	0.92	47.5
415200	4 066	139 000	1.17	34.2

Table 2.1: Catchment areas and annual runoff characteristics for the Wimmera River at Eversley (415207), Glynwylln (415206), Glenorchy (415201) and Horsham (415200).

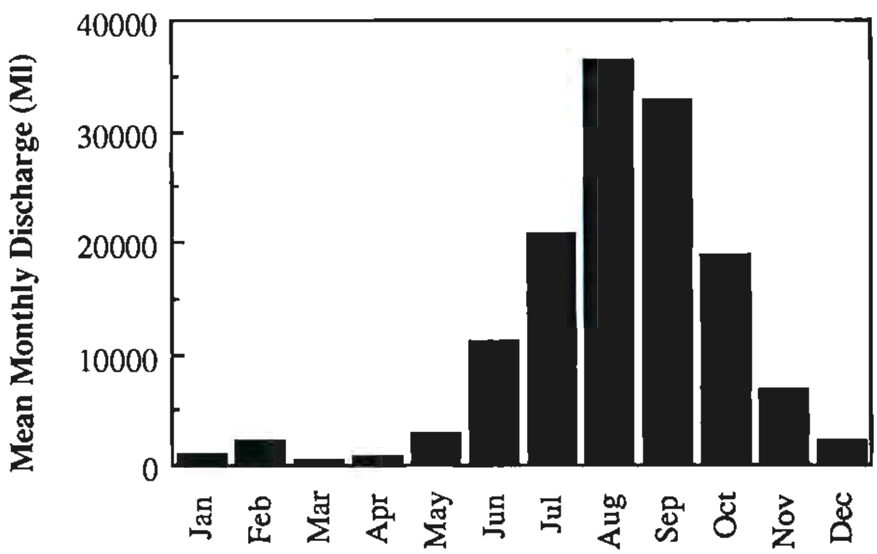


Figure 2.3: Seasonal variation of mean monthly discharge in the Wimmera River at Horsham.

2.3.2 SALINITY REGIME.

Hooke (1991) examined salinities and salt fluxes in the Wimmera River catchment for the period 1976 to 1989. Salinities are generally low in streams in the Grampians and on the upper slopes of the Pyrenees Ranges. On the Wimmera Plains stream salinities increase sharply. In the middle to lower catchment stream salinities are modified to varying degrees by interaction with the WMSDS. There is generally an inverse relationship between flow and salinity therefore, given the seasonal nature of flows in the catchment, stream salinities tend to be lower during the late winter and spring.

Table 2.2 provides median and flow weighted mean salinities and mean annual salt fluxes for the Wimmera River. The difference between the flow weighted

Gauging Station	Flow Weighted Mean Salinity (EC units)	Median Salinity (EC units)	Annual Salt Flux (t/a)
415207	390	1400	4 800
415206	430	2400	19 000
415201	420	920	25 400
415200	300	1000	17 200

Table 2.2: Salinity characteristics and salt loads for the Wimmera River at Eversley (415207), Glynwylln (415206), Glenorchy (415201) and Horsham (415200).

mean and median salinities provides an indication of the variation of salinity with flow. The reduction in annual salt flux between Glenorchy and Horsham is due to diversion to the WMSDS. It was noted by Hooke that significant groundwater inflows occur downstream of Horsham and that the annual salt flux may double between Horsham and Lochiel and then double again before the Wimmera River enters Lake Hindmarsh. Hooke (1991) also calculated annual salt fluxes per unit area for some streams. The median annual salt flux per unit area for unregulated streams in the Wimmera River Catchment for Hooke's study period was 14.5 $t/a.km^2$.

Since the salinities in the Wimmera River are relatively high, large changes in salinity can occur during flow events. If these changes are too rapid they can have significant environmental impacts and may even lead to the death of fish (Anderson and Morison, 1989c). Another important aspect of the groundwater inflows to the lower Wimmera River is the occurrence of density stratified pools or saline pools (Anderson and Morison, 1989c; 1989g). The water below the halocline in saline pools is typically anoxic, acidic and contains hydrogen sulphide. This renders the water unsuitable for fish and other aerobic aquatic fauna (Anderson and Morison, 1989c; 1989g).

2.4 SALINE POOLS: ANDERSON AND MORISON'S STUDY.

A detailed assessment of the aquatic environment in the Wimmera River and its tributaries was performed by Anderson and Morison (1989a-f) between 1986 and 1989. Anderson and Morison (1989f; 1989g) provide summaries of this study. From the perspective of the current study, the most significant discovery made by Anderson and Morison was that extensive density stratification associated with accumulation of saline water in scour depressions exists in the Wimmera River downstream of Roseneath. Pools where the salinity difference between the surface water and that near the stream bed exceeded 1 000 EC were referred to as

Survey	Period	Flow Condition prior to survey.
1	29/4/86-11/7/86	Discharge at Horsham had been < 5 Ml/d for 141 days on the 29/4/86 (prolonged low flow).
2	10/7/86	Peak flow of 190 Ml/d at Horsham on the 4/7/86. Peak flow of 700 Ml/d (estimated) at Dimboola on the 9/7/86.
3	1/8/86-5/8/86	Peak flow of 3 130 Ml/d at Horsham on the 26/7/86.
4	23/2/87-3/3/87	Generally low since November 1986 with 2 small flow events (peak discharge < 350 Ml/d at Horsham).
5	30/3/86-5/4/86	Environmental Release: 100 Ml/d for 25/2/87-3/3/87 50Ml/d for 4/3/87-10/4/87

Table 2.3: Dates of and flow condition prior to Anderson and Morison's (1989c) water quality surveys. Note: the flow event prior to Survey 2 had not reached sites (on the lower river) surveyed at the end of Survey 1.

saline pools. Anderson and Morison (1989c; 1989g) conducted an extensive field study of this stratification which is now discussed.

Anderson and Morison (1989c) conducted detailed water quality surveys on five occasions during 1986 and 1987. Vertical profiles of salinity, temperature and dissolved oxygen were measured in pools at irregular intervals along the Wimmera River. Unfortunately Anderson and Morison measured their profiles relative to the stream bed at the deepest point in the pool. They relied on this elevation being constant and on being able to relocate the deepest point for each measurement. There is therefore some uncertainty in the vertical datum of each profile which makes detailed comparison difficult. However most of the observed changes are so significant that the general conclusions drawn by Anderson and Morison (1989c; 1989g) are well supported by the data. Table 2.3 provides the dates of the surveys and the flow conditions prior to each survey.

During their surveys Anderson and Morison (1989c; 1989g) observed groundwater seepage entering the Wimmera River Channel at many locations along the river, particularly downstream of Roseneath. Density stratification was also observed in at least some pools during every survey. Seepage of groundwater was observed to be localised along many reaches of the river; however inflows of groundwater to the river occurred over extensive areas downstream of Antwerp (Figure 2.1).

Survey 1 was conducted during winter after an extended period of low flow and saline pools were found extensively downstream of Roseneath. The saline pools typically consisted of a saline layer below a fresh layer. The saline layer was up to 5° C warmer than the surface layer and was anoxic. In most saline pools the halocline, thermocline and oxycline coincided. In shallow parts of the river the saline layer often extended close to the surface while in deeper areas the saline layer was usually 2-3 m deep.

A small flow event occurred during July 1986 following which five sites were resurveyed. Little or no change in the location of the halocline and no change in the quality of the water below the halocline was observed at four of the sites. At site 213 (referred to as Big Bend in Chapter 8) a dramatic change occurred. Prior to the flow event the vertical salinity profile at site 213 was almost uniform and the salinity was approximately 12 000 EC. After the event the salinity in the bottom 6 m of water had risen to 25 000 EC and that in the upper 3 m had decreased to 5 000 EC. A significant volume of saline water had been observed in shallow anabranch channels and backwater pools immediately upstream of this site prior to the flow event. Anderson and Morison (1989c; 1989g) concluded that this saline water had been carried downstream and deposited at site 213 at the start of the event and that relatively fresh water displaced the upper 3 m of water during the event.

At the end of July a major flow event occurred in the river after which Survey 3 was conducted. Most saline pools were completely flushed by this event. When Survey 4 was conducted at the start of 1987 saline pools had returned at most sites. The salinity and temperature of the water below the halocline was similar to that during the first survey. In addition to the saline pools, thermal stratification was observed in the upper layer which was now warmer than the saline layer. This thermal stratification was associated with widespread deoxygenation in the upper layer.

An experimental environmental release was conducted following the fourth survey. For the first week a discharge of 100 Ml/d was released upstream of Horsham. The dishcarge was subsequently reduced to 50 Ml/d. The experimental release resulted in destruction of the thermal stratification in the upper layer and a significant increase in dissolved oxygen. However the saline pools were unaffected except in shallow areas where significant mixing occurred.

Anderson and Morison (1989c; 1989g) concluded that, except at shallow locations, saline pools were stable to flows less than 700 Ml/d but were flushed by flows of 3 000 Ml/d. Furthermore, given the flow regime in the Wimmera River, saline pools would exist for large periods of the year. However the fact that a flow event with an estimated peak discharge of 700 Ml/d did not cause a large volume of saline water to be flushed from the saline pools does not mean that a flow rate of 700 Ml/d is not associated with a significant mixing rate. It is possible that the duration of the high flow was short and therefore only a small volume of saline water was flushed from the saline pools. On the other hand the experimental release was sustained for more than one month during which time little mixing occurred. It is therefore possible to conclude that discharges of 50 Ml/d do not cause significant mixing.

Anderson and Morison (1989c; 1989g) also considered the formation of saline pools. It was argued that in many cases the saline pools formed directly from groundwater seeping into the river channel and collecting in scour depressions. The following observations support this conclusion. Saline groundwater was observed to be seeping into the channel at many locations. Saline pools regularly formed at the same locations and the salinity and temperature of the saline layer was consistent between stratification episodes. Water samples also showed that the ionic composition of the water in the saline pool and the groundwater were similar. However there were locations such as site 213 where observations of salinity changes during flow events indicated that saline water could also be transported from the upstream river channel and deposited in scour depressions. This saline water may have originated from groundwater seeping into the upstream channel.

Anderson and Morison (1989c; 1989g) argued that saline pools were an ecologically significant phenomenon because of the water quality associated with the saline layer. Many inland freshwater riverine faunal and floral communities in the Murray-Darling Basin are likely to be tolerant of low to moderate salinities. However the anoxic conditions associated with saline pools would render the water below the halocline uninhabitable by aerobic aquatic organisms. This means that a significant proportion of the river bed and banks would be unusable. Furthermore branches and logs on the river bed which provide important cover and spawning and feeding sites for fish and habitat for benthic invertibrates would be inaccessible (Anderson and Morison, 1989c; 1989g).

Anderson and Morison's (1989c; 1989g) study identified the existence of saline pools in the Wimmera River and indicated that they are ecologically significant. It also identified groundwater inflows as the primary cause of the stratification. However details of the processes and rates saline pool formation and the flow rates and processes which cause significant mixing remained unclear. The current study has examined these issues and has improved our understanding of the behaviour of saline pools.

2.5 DATA ISSUES.

Several different types of data have been used in various parts of this study. These consist of data collected specifically for this project and data collected as part of routine stream and channel monitoring by the RWC.

2.5.1 EXISTING DATA.

2.5.1.1. Hydrographic Stream Gauging Records.

The RWC operate a stream gauging network throughout the Wimmera River Catchment. The locations of relevant gauging stations are provided on Figure 2.1. Table 2.4 summarises the data collected at each monitoring site. Sites are included in Table 2.4 on the basis that data from that site has been used at some stage in this study. In some cases new instrumentation, which is not included in the table, has been added recently. In other cases data has not been available for the entire period required for the specific analysis in which case the fact has been noted when the analysis has been described.

Stage and salinity data are collected by the RWC hydrographic section using standard hydrographic practices. Where stage and/or salinity data are listed as being available in Table 2.4, they are continuously monitored data. Where flow data are available they have been calculated from the monitored stage using a rating curve which is maintained via regular flow gauging measurements made by the RWC hydrographic section. Electrical conductivity at the standard temperature of 25° C is used as a measure of stream salinity.

Before any of the continuously monitored data were used they were checked for errors as follows. The data to be used were plotted as time-series and inspected visually. This allowed obvious errors such as missing data, sudden jumps in discharge or reversed hydrographs to be located. Salinity data were also compared with flow data. Several problems with unreliable salinity data were identified and referred to the RWC hydrographic section where the data were either corrected or the quality codes were amended. A significant number of salinity data were either missing or unreliable due to equipment failures. The system of quality codes used by the RWC hydrographic section was used as the basis for inclusion or exclusion of specific salinity data in the analyses performed.

2.5.1.2 Other Salinity Records.

Salinity samples are collected at gauging stations by RWC hydrographers on every occasion that a stream discharge measurement is performed for rating the

Gauging Station	Name	Control	Stage	Flow	Salinity
415200	Wimmera R at Horsham	LFW	yes	yes	yes
415201	Wimmera R at Glenorchy	LFW	yes	yes	no
415203	Mt William Ck at Lk. Lonsdale	LFW	yes	yes	no
415206	Wimmera R at Glynwylln	LFW	yes	yes	yes
415207	Wimmera R at Eversley	LFW	yes	yes	no
415223	Burnt Ck at Wonwondah East	LFW	yes	yes	no
415237	Concongella Ck at Stawell	LFW	yes	yes	no
415238	Wattle Ck at Navarre	LFW	yes	yes	no
415239	Wimmera R at Drung Drung	natural	flood	flood	no
415246	Wimmera R at Lochiel	natural	yes	yes	no
415247	Wimmera R at Tarranyurk	natural	yes	no	yes
415251	McKenzie R at McKenzie Ck	LFW	yes	yes	no
415256	Wimmera R at Upstream of Dimboola	natural	yes	yes	yes
415258	Wimmera R at Upstream of Glenorchy	natural	no	no	yes

Table 2.4: Hydrographic stream gauging station operated by the RWC in the Wimmera River catchment from data has been used in this study. Note: LFW refers to a flow control consisting of a fixed crest weir which becomes drowned at high flows.

stream gauge. These data provide an irregular, non-random series of stream salinities. The data are non-random because a special effort is made to obtain stream gauging measurements during high flows so that the stage-discharge rating curve for the gauging station can be established properly. These data are referred to as hydrographic spot salinity data in the remainder of this thesis.

Salinity samples are also collected every month by RWC hydrographers as part of the Victorian Water Quality Monitoring Network (VWQN). These data provide an unbiased set of stream salinity data. VWQN spot salinity data from the Wimmera River at Glynwylln (415206) and Glenorchy (415201) and the Mt William Creek at Lake Lonsdale (415203) have been used in this study.

2.5.2 WMSDS FLOW DATA.

Flow data for the WMSDS are available as daily or weekly totals. These are estimated from daily flow readings taken by RWC water rangers. These data could not be checked by visual inspection since the regulated nature channel flow tends to remove any natural patterns from the flow variation. All the WMSDS flow data used were checked against the original paper records by the RWC (J.

Martin, RWC, Per. Com.). It is believed that the WMSDS channel flow data are significantly less accurate than the stream discharge data collected by the RWC hydrographic section due to the different collection techniques.

2.5.3 COLLECTION OF ADDITIONAL DATA.

The adequacy of the stream monitoring network in the Wimmera River Catchment for this project was assessed at the beginning of this project. As a result of that assessment additional stream salinity data were collected by the RWC for the Wimmera River at Glynwylln and Horsham. While collection of much more stream discharge and salinity data would have been desirable, financial considerations precluded this.

2.5.4 DATA COLLECTION AT SALINE POOLS.

A significant amount of data was collected at a number of saline pools during field trips for this project, by the Kaiela Fisheries Research Station (KFRS) (T. Ryan, KFRS, Per. Com.) and by the RWC hydrographic section. These data are described in detail in Chapter 8.

2.6 SUMMARY.

The Wimmera River and its catchment have been described in this chapter. Rising in the Pyrenees Ranges, the Wimmera River receives tributary inflows from streams flowing from both the Pyrenees and Grampians Ranges and flows into a series of terminal Lakes the first of which is Lake Hindmarsh. It is characterised by a highly variable flow regime. Stream salinities are considered to be high and are of significant concern. Large groundwater inflows lead to the development of saline pools which have significant environmental impacts. The processes controlling the behaviour of these saline pools were poorly understood at the beginning of this study. The discharge and salinity data used in this project were also described.

CHAPTER 3 - OPEN CHANNEL FLOW: THEORY AND MODELLING.

This chapter provides an introduction to the theory of river hydraulics and solute transport. Four models capable of simulating water and solute routing in natural streams are briefly discussed. The MIKE 11 (DHI, 1992a, 1992b) model, which was chosen for use in this project, is then discussed in detail.

Introductions to the fundamental concepts of Open Channel Hydraulics are available in standard texts (eg Chow, 1959; Henderson, 1966; French, 1986) and these have been used as the basis for the initial discussion. A basic understanding of fluid mechanics is assumed.

3.1 CHANNEL AND FLOW CLASSIFICATION.

An open channel flow is a flow in a conduit which has a surface exposed to the atmosphere (Chow, 1959; French, 1986). Such a surface is referred to as a free surface and its existence introduces important gravitational phenomena into the flow. Open channel flows can be classified in terms of channel type and flow variations with respect to space and time.

Channels can be differentiated on the basis of whether they are natural or artificial. Artificial channels tend to have a uniform cross-section and to include structures which enable the flows in the channel to be controlled. Conversely, natural channels tend to have irregular cross-sections and to be free of artificial controls. While the principles of open channel hydraulics apply to both natural and artificial channels, the understanding of flows in natural channels requires a knowledge of fields such as hydrology, geomorphology and sediment transport (French, 1986). Channels can be classified on the basis of the regularity of their cross-section and bed slope. A channel with a constant cross-sectional shape and bed slope is called a prismatic channel (French, 1986). Natural channels are typically non-prismatic or irregular (Richards, 1982).

Open channel flows are said to be steady if the depth is not changing over time and unsteady when the depth changes with time. This is a relative concept since observers in different reference frames may classify the same flow differently. French (1986) uses the example of a surge to illustrate this point. A stationary observer seeing a surge propagate along a channel would classify the flow as unsteady while an observer travelling with the surge would not observe any change in depth and would therefore classify the flow as steady.

Flows which do not vary in space are referred to as uniform flows. The release of gravitational potential energy is exactly balanced by energy losses to flow friction in a uniform flow. In non-uniform flows the depth varies in space (French,1986). This variation may be gradual or rapid. In rapidly varied flows, such as in hydraulic jumps, the depth changes significantly over a relatively short distance. Gradually varied flow is characterised a depth which changes only slowly over a relatively long distance (French, 1986). Gradually varied and rapidly varied flows are really the extreme cases of a continuum of possible steady open channel flows; however open channel flows are typically treated as either gradually or rapidly varied. Mathematically these two flow types are differentiated by the use of different assumptions (Yevjevich, 1975).

Natural rivers are characterised by irregular channels (Richards, 1985) and unsteady, non-uniform flow (Yevjevich, 1975). Where flows in natural river systems are driven by rainfall processes the resultant flood waves in the river are gradually varied (Henderson, 1966; Yevjevich, 1975; French, 1985). The gradually varied open channel flow equations is provided in §3.3. However at specific locations in an open-channel, such as at weirs, the flow may become rapidly varied. Wherever rapidly varied flow is encountered an alternative (to that used for gradually varied flow) mathematical description applies.

3.2 THE IMPORTANCE OF VISCOSITY AND GRAVITY.

The effects of viscosity and gravity on an open channel flow depend on the Reynolds Number, Re, and the Froude Number, Fr, respectively. Reynolds number is given by $Re = ul/\nu$, where u is an appropriate velocity scale, l is an appropriate length scale and ν is the kinematic viscosity. The Reynolds number represents the ratio of the inertial to viscous forces. Flow depths and velocities in natural streams are usually such that flow is fully turbulent (French, 1986) in which case the Reynolds Number plays only a limited role (Henderson, 1966).

Conversely, the Froude number is of primary importance since open channel flow is a free surface phenomenon (Henderson, 1966). The Froude number is given by $Fr = U/\sqrt{gD}$, where $D=A/B$ is the hydraulic depth, A is the cross-sectional area and B is the water surface width. Fr represents the ratio of the inertial forces to the gravitational forces. Flows can be classified as subcritical ($Fr < 1$), critical ($Fr = 1$) and supercritical ($Fr > 1$) on the basis of the Froude Number. Gravitational forces dominate for subcritical flow and inertial forces dominate for supercritical flow.

Subcritical and supercritical flows can also be interpreted in terms of the celerity of an elementary gravity wave, $\sqrt{gD}$. For supercritical flow, the velocity of the flow is higher than the celerity of a gravity wave hence it is not possible for a disturbance to propagate upstream. Conversely, if the flow velocity is less than the wave celerity, a wave can propagate upstream, $Fr < 1$ and the flow is subcritical. When the flow is critical the wave celerity and the water speed are identical and a wave trying to propagate upstream will become a standing wave (Yevjevich, 1975; French, 1986). Subcritical flows become supercritical in a smooth transition through the critical flow condition; however supercritical flows become subcritical in a hydraulic jump. A hydraulic jump is characterised by a sudden increase in the water level downstream, a large amount of turbulence and a significant energy loss. Flows in natural streams are usually subcritical (Henderson, 1966).

3.2.1 THE HYDRAULIC CONTROL CONCEPT.

Henderson (1966) defines a hydraulic control as "any feature which determines a depth-discharge relationship". Any channel feature, such as a weir, a rise in the bed or narrowing of the channel, which induces critical flow is a hydraulic control because there is a unique relationship between the depth and discharge when flow is critical. Such a feature is referred to as a choke. Downstream of a choke the flow will be supercritical. Any disturbance in supercritical flow will be swept downstream since the flow velocity is greater than the wave celerity. Therefore the flow can only be influenced by upstream features and is said to be subject to upstream control. Conversely the flow upstream of a choke is subcritical, is influenced by downstream channel features and is subject to downstream control.

Other features can act as hydraulic controls. For example a subcritical flow entering a lake is controlled by the level of water in the lake (provided the lake is not overflowing). Uniform flow also acts as a hydraulic control because the water level is being determined by a local balance between the release of gravitational potential energy and the loss of kinetic energy to heat through flow friction.

In natural streams the concept of a hydraulic control becomes more difficult to apply. Because natural channels are irregular, uniform flow does not exist and thus is not a hydraulic control. Often the flow in a natural channel remains subcritical, therefore chokes do not act as hydraulic controls. If the flow remains subcritical until it reaches the sea, the true hydraulic control is the sea level. In subcritical flow the effects of a disturbance propagate upstream all the time becoming smaller with distance from the disturbance (Henderson, 1966).

Features such as narrow points in the channel or high points in the channel bed which constrict the flow produce backwater effects. If the constriction is sufficiently far upstream of the hydraulic control, the backwater effect from the true hydraulic control will have become small and the local constriction may be the most important determinant of the upstream water level. In practice such features are sometimes referred to as controls even though they are not hydraulic controls in the strict sense. For example at gauging stations a cross-section causing significant backwater effects may be identified as the control for that gauge even if the flow remains subcritical. Of course in many natural channels a series of features will exist that are causing backwater effects and contributing to the determination of the water level. Identification of one dominant feature may be difficult. It should be noted that flow resistance also plays an important role in determining the local water level even though the flow is non-uniform.

3.3 THE GRADUALLY VARIED UNSTEADY FLOW EQUATIONS.

The gradually varied shallow water flow equations for one-dimensional channel flow were originally derived by de St Venant and can be found in any open-channel flow text. These equations express the continuity and momentum principles as they apply to open channel flows and are based on the following assumptions (Cunge et al.; 1980).

(1) The flow is one-dimensional. This implies that the velocity is uniform across the cross-section.

(2) Hydrostatic pressure exists. This implies that vertical accelerations are negligible compared with g.

(3) The effects of boundary friction and turbulence can be accounted for by resistance laws which are analogous to those used for steady flows.

(4) The longitudinal bed slope is sufficiently small for the cosine of the angle it makes with the horizontal to be replaced with unity.

The St Venant equations for an irregular channel are:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\left(\frac{\partial y}{\partial x} - S_o\right) + gAS_f = 0\,, \qquad (3.1a)$$

$$\frac{\partial y}{\partial t} + \frac{1}{B}\frac{\partial Q}{\partial x} = 0\,. \qquad (3.1b)$$

In Equation 3.1 Q is the discharge, A is the cross-sectional area, y is the water depth, B is the water surface width, S_f is the friction slope and S_o is the bed slope. Distance and time are represented by x and t respectively and g is the gravitational acceleration.

Equation 3.1a is a statement of Newton's Second Law (force = rate of change of momentum) and is referred to as the dynamic equation. The terms on the left hand side of the dynamic equation represent, from left to right, the local acceleration, the convective acceleration, the horizontal pressure gradient resulting from the water surface slope and the force arising from flow resistance. All the terms in Equation 3.1a can be determined from the flow and channel geometry with the exception of the flow resistance. An empirical flow resistance relationship is usually used to specify S_f (Cunge et al., 1980). Equation 3.1b is the continuity equation. The St Venant equations can be solved numerically provided the initial conditions and two boundary conditions are known. For subcritical flow one boundary condition is required at each of the upstream and downstream extremities of the open channel (Cunge et. al, 1980).

3.3.1 FLOW RESISTANCE.

As a result of flow resistance, energy is being continually dissipated in the flow of a real fluid (Henderson, 1966). In this Thesis the Manning equation (Equation 3.4) is used to describe flow resistance.

$$U = \frac{R^{2/3} S_f^{1/2}}{n} \qquad (3.4)$$

In Equation 3.4, R is the hydraulic radius, S_f is the friction slope or the slope of the total energy line, and n is a roughness parameter known as Manning's n. It is noted that n has the dimension $m^{-1/3}s$.

The value of Manning's n depends on the boundary roughness characteristics of the open channel. Tabulated values of n are available for different materials (eg. Chow, 1959; Henderson 1966) and photographs of different open channels with typical values of n are provided in Chow (1959) and French (1986). However flow resistance in natural streams channels depends on many factors including: channel substrate, bed forms, bends, secondary currents, obstructions including large woody debris, expansion and contraction losses (French, 1986), vegetation, channel irregularity, channel alignment, silting and scour, size and shape of channel, stage and discharge (Chow, 1959). There is a significant range in Manning's n quoted for natural channels (Henderson, 1966). Here n should be thought of as a flow resistance parameter because it includes effects in addition to boundary roughness. In practice appropriate values of n are determined in hydraulic models through a process of calibration (Cunge et al., 1980). Furthermore it is assumed that the Manning equation, which was developed on the basis of experiments on steady uniform flows, is an appropriate description of flow resistance in unsteady nonuniform flows (Cunge et al., 1980).

3.3.2 STORAGE AND FLOW ROUTING.

Henderson (1966) provides an enlightening discussion of the processes controlling flow routing in natural channels. Generally the terms in the dynamic equation are such that $gA\,S_o > gA\,\frac{\partial y}{\partial x} > \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) > \frac{\partial Q}{\partial t}$ and these are balanced by gAS_f. If gAS_o is the only significant term then the dynamic equation reduces to $S_o = S_f$. Waves described by the dynamic equation disappear and that the depth depends only on the discharge. While this simplification eliminates waves described by the dynamic equation another form of wave remains. The continuity equation can be rearranged as:

$$\frac{dQ}{dy}\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = 0 \qquad (3.5)$$

or

$$\frac{dy}{dt} = \frac{dx}{dt}\frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0 \qquad (3.6)$$

where

$$\frac{dx}{dt} = c = \frac{1}{B}\frac{dQ}{dy} \quad . \qquad (3.7)$$

Equations 3.6 and 3.7 describe a kinematic wave which moves with celerity c. It is noted that this kinematic wave only moves downstream and thus cannot describe any backwater effects. It can be seen from Equation 3.7 that the celerity of the kinematic wave is determined by the change in storage associated with a change in flow (Henderson, 1966).

Given that S_o is usually the most significant term in the dynamic equation, the main bulk of a flood wave moves as a kinematic wave (Henderson, 1966). However the other three terms in the dynamic equation may not be negligible in which case dynamic waves will move upstream (assuming subcritical flow) and downstream from the kinematic wave. Provided that $Fr < 2$ these dynamic waves attenuate (unless the flow rises very rapidly which is unlikely in a natural flood) (Henderson, 1966). The dynamic terms, particularly $\frac{\partial y}{\partial x}$, act to modify the kinematic wave. Specifically the dynamic terms introduce dispersion and subsidence (which may be appreciable) and lead to a second order modification of the shape and speed of the kinematic wave (Henderson, 1966). Henderson notes that for slowly rising waves, such as natural flood waves, substantial departures from the kinematic wave celerity can only be produced by storage effects such as off-channel storage.

It was stated above that the order of importance of the terms in the dynamic equation was $gA\,S_o > gA\frac{\partial y}{\partial x} > \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) > \frac{\partial Q}{\partial t}$ and that these terms are balanced by gAS_f. The relative magnitude of these terms is now considered in more detail. For a wide rectangular channel $\frac{\partial y/\partial x}{S_o} \propto S_o^{-5/3}$, thus $\partial y/\partial x$ becomes small for steep streams. The local and convective acceleration terms are of the same order and the ratio of the convective acceleration to $\frac{\partial y}{\partial x}$ is of the order Fr^2. Thus the $\frac{\partial y}{\partial x}$ term is of equal or larger order than either the local or convective acceleration terms unless $Fr >> 1$ which only occurs is very steep streams. For streams with a gentle slope $Fr << 1$ and the acceleration terms become negligible; however $\partial y/dx$ may become significant. It is conceivable that there are intermediate values of slope for which all four terms may be significant (Henderson, 1966).

Given that some of the terms in the dynamic equation may be insignificant in certain circumstances, this equation has often been simplified. The two most common simplifications involve neglecting the acceleration terms and neglecting the acceleration and $\partial y/\partial x$ terms. Models based on these simplifications are usually referred to as diffusive wave models and kinematic wave models respectively. Diffusive wave models assume that the friction slope is equal to the water surface slope while kinematic wave models assume that the friction slope is equal to the bed slope.

3.3.3 VIOLATION OF THE ST VENANT ASSUMPTIONS.

In some applications various supplementary terms and coefficients are added to the St Venant equations in order to relax the assumptions used in their derivation. In natural streams inundation of over-bank areas can contribute significantly to the storage of water but not to the discharge. This phenomenon is often modelled by ignoring the over-bank part of the cross-section when calculating the geometric variables in the dynamic and resistance equations but using the total water surface width to specify the storage width in the continuity equation (Cunge et al., 1980).

In many applications lateral inflows are incorporated in which case the continuity equation is modified to:

$$B\frac{\partial y}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{3.8}$$

where q is the lateral inflow per unit length. Usually the lateral inflow is ignored in the dynamic equation (Cunge et al., 1980).

The dynamic equation is traditionally derived using the momentum principle and it is therefore a vector equation. An implicit assumption is made that the channel is straight. However perhaps the most striking single feature of a natural stream is the meander pattern. Therefore one of the most fundamental assumptions used to derive the St Venant equations is violated. Discussion of this issue in the literature is notable for its absence.

3.4 SOLUTE TRANSPORT IN OPEN CHANNEL FLOWS.

Solutes are transported in open channel flow by advection and by various mixing processes. Advection simply refers to the transport of solute by an imposed current (Fischer et al, 1979). Although the term convection is often used to describe advection in open channel flows the process is not a convective process. Convective transport is transport in a flow which results from the action of gravity upon a density anomaly in the fluid, for example flow over a heated surface. Convective transport can influence the flow by redistributing the density anomaly thus the flow and transport are intimately linked (Fischer et al., 1979).

Mixing in open channel flows occurs by two distinct processes: diffusion and dispersion. Molecular diffusion is the scattering of particles by random molecular motions which can be described using Fick's law and the classical diffusion equation (Fischer et al., 1979). Molecular diffusion is only effective over short distances and long times (Tennekes and Lumley, 1972). Turbulent diffusion is the scattering of particles by random turbulent motions in a fluid. Turbulent diffusion is often considered to be analogous to molecular diffusion but acts much more rapidly and over much longer length scales (Fischer et al., 1979).

Dispersion results from the combined action of a velocity gradient or shear and transverse (at right angles to the flow) mixing. Because of the role of velocity shear the dispersion process is often referred to as shear flow dispersion. The classic analyses of dispersion are those by Taylor (1953) for laminar flow in a pipe and Taylor (1954) for steady turbulent flow in a pipe. When a velocity gradient is present two elements of fluid, initially side by side in the flow, will be gradually

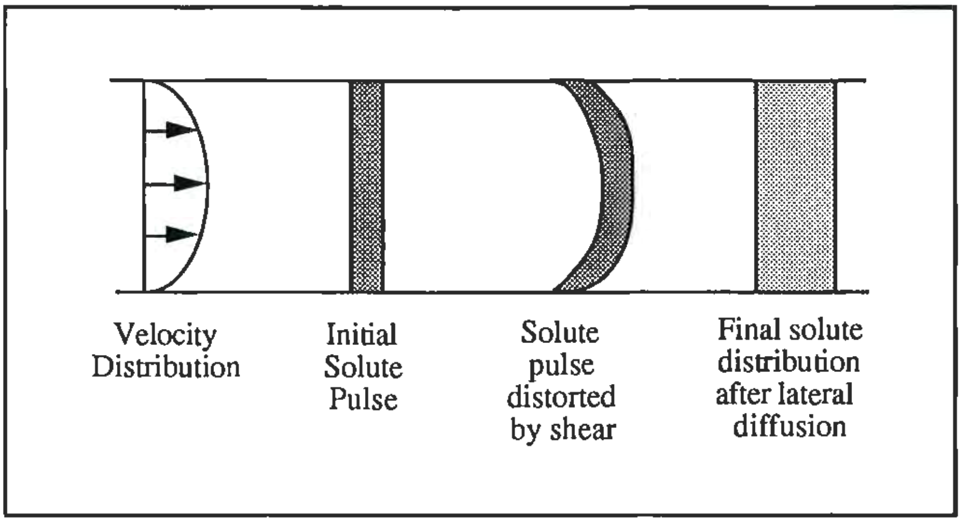


Figure 3.1: The longitudinal dispersion of a pulse of solute by a shear flow in a pipe or open channel. The pulse is distorted by the velocity shear and mixed laterally by turbulent diffusion. As a result longitudinal mixing occurs.

separated by the effects of the shear. If the fluid in these elements is subsequently mixed across the flow, a longitudinal spreading of any solute originally contained in the two elements will have occurred. This is illustrated in Figure 3.1. This combination of longitudinal stretching due to the velocity shear and transverse mixing by, in the case of turbulent flow, turbulent diffusion is the longitudinal dispersion process.

Taylor argued that, provided a solute was fully mixed across the flow and the flow was statistically steady, the one-dimensional Advection-Dispersion equation was an accurate description of shear flow dispersion. The one-dimensional Advection-Dispersion equation is:

$$\frac{\partial C}{\partial t} + U\frac{\partial C}{\partial x} = D\frac{\partial^2 C}{\partial x^2}, \qquad (3.9)$$

in which C is the cross-sectional mean concentration and D is the coefficient of dispersion. While Taylor's analyses applied to pipes, similar results can be obtained for wide prismatic channels (Elder, 1959) and straight rivers (Fischer, 1967).

3.4.1 DISPERSION IN NATURAL STREAMS.

Flows in real streams are far removed from steady flows in straight prismatic channels. Stream channels contain bends, sandbars, side pockets, pools, riffles and many other irregularities all of which contribute to the dispersion . While some streams may be so irregular that Taylor's analysis is inapplicable, for example a mountain pool and riffle stream; the majority of streams are sufficiently uniform for an approximate analysis (Fischer et al., 1979). Limitations of such an analysis must be considered.

The Advection-Dispersion equation does not apply until a solute is completely mixed across the cross-section. An estimate of the distance required for complete mixing of a solute released at the side of the channel (by say a tributary stream) is given by $L_m=0.4UB^2/\varepsilon_t$ where B is the channel width and ε_t is the transverse turbulent diffusion coefficient (Fischer et al., 1979). Empirical results show that ε_t/du^* is in the range 0.3 - 0.7 (Fischer et al., 1979), where d is the depth, u* is the shear velocity which is given by $\sqrt{gRS_f}$ and R is the hydraulic radius. Suitable estimates of the above variables for the Wimmera River are U=0.5 m/s, B=40m, $R \approx d$=4m and S_f=0.0005 in which case L_m is of the order of 1 000 m which is short compared to the length of stream of interest. Therefore, with the exception of saline pools, the assumption that solutes are completely mixed across the cross-section can be made.

An interesting phenomenon that is often observed in dispersion tests in streams is the development of a long tail of solute. Dispersion tests are conducted by releasing dye into the stream and monitoring how it spreads longitudinally as it progresses downstream. It is believed that the tail results from transient storage of solute caused by irregularities in the stream (Fischer et al., 1979). The one-dimensional Advection-Dispersion equation is not capable of simulating this process.

Nordin and Sabol (1974) examined data from 51 dispersion tests conducted. If the one-dimensional Advection-Dispersion equation holds and sufficient time has elapsed time has elapsed for the longitudinal distribution of solute to become Gaussian, the peak concentration of a solute cloud should decrease with the square root of time and the variance of the solute cloud should increase linearly with time. Nordin and Sabol (1974) observed systematic deviations from this theoretical behaviour in many of the tests they examined. They then suggested that either the one-dimensional theory was inadequate or the mixing time (or distance) required before the one-dimensional theory was applicable was much longer than generally supposed.

The existence of solute tails and the deviations from one-dimensional advection-dispersion theory have motivated some workers to develop a theory incorporating transient storage or dead zones (eg. Thackston and Schnelle, 1970; Sabol and Nordin, 1978; Nordin and Troutman, 1980; Bencala and Walters, 1983; Denton, 1990; Seo and Maxwell, 1992). The transfer of solute between the storage zones and the main flow is assumed to be proportional to the concentration difference.

Nordin and Troutman (1980) examined the skewness of solute clouds expected on the basis of the advection-dispersion theory and transient storage theory. Both predict that skewness decreases with time but the skewness is larger when transient storage is included. Interestingly Nordin and Troutman found that skewness was almost constant over time when they examined data from four different dispersion tests. This contradicts the predictions of both theories. Bencala and Walters (1983) and Seo and Maxwell (1992) found that models incorporating transient storage were significantly better at predicting observed concentration profiles than models based on advection-dispersion theory.

While transient storage theory appears to be superior to advection-dispersion theory neither is in full agreement with field observations. Bencala and Walters (1983) note that storage zones of stagnant water are easy to envision but that " it is not easy to envision a linear physical driving mechanism that simultaneously transfers mass between the stream and the storage zone, distributes it uniformly throughout the storage zone and yet prevents the storage zone from moving longitudinally." It may be concluded that details of the longitudinal mixing processes in streams remain unclear. In either case longitudinal mixing only accounts for only a small proportion of the downstream transport of a solute when longitudinal concentration gradients are small (or the length scale associated with longitudinal variations in concentration is sufficiently large) (Fischer et al., 1979). In practice the one dimensional advection-dispersion theory is often used (Chatwin and Allen, 1985). The Advection-Dispersion equation can be solved numerically provided the upstream concentration, advective velocity and the dispersion coefficient are available.

The magnitude of the dispersion coefficient in Equation 3.9 varies considerably. Results of various field measurements of longitudinal dispersion are collated by Fischer et. al. (1979) and show a range of three orders of magnitude. Fischer et al. (1979) suggest that, since the exact effects of irregularities are unknown, it makes little sense to aim for too high an accuracy when predicting the dispersion coefficient. Furthermore most applications are insensitive to the exact value of dispersion coefficient. An approximate estimate for the dispersion coefficient is (Fischer, 1975; Fischer at al., 1979):

$$D = 0.011 U^2 B^2 / d u^* \, . \qquad (3.10)$$

3.5 MODELLING WATER AND SOLUTE ROUTING.

It is not the intention of this section to present an exhaustive review of stream modelling. Rather the section provides an overview of the more commonly used approaches to stream modelling and the reasons for choosing the model that was used for this study. Theoretical equations for one-dimensional open channel flow and for solute transport in one-dimensional open channel flow were presented in §3.3 and §3.4.

The St Venant and Advection-Dispersion equations can be solved by various numerical techniques (Sobey, 1984). Such solutions form the basis of many stream models. These models are often referred to as physically based models because they solve the partial differential equations which are purported to describe the physical processes of importance in a stream. The numerical solution techniques used to solve the partial differential equations are not discussed here; however the solution schemes used in the MIKE 11 model are an example of such schemes and are discussed in §3.6 and Appendix 1. It is noted that accurate solution of the Advection-Dispersion equation is difficult; however accurate numerical schemes for its solution do exist (Sobey, 1984; Verwey, 1994). Use of the term "physically based" in this thesis is not intended to indicate any inherent superiority since it is believed that different types of stream models are suited to different types of problems. Selection of an appropriate model is a critical part of any modelling study.

Physically based numerical models of stream flow and solute transport provide approximate solutions to the governing equations at a finite number of locations or computational points which are used to represent the system being modelled. The physical characteristics of the system are described by specifying cross-sections and any parameters, such as the resistance coefficient or the dispersion coefficient, which are required at each computational point. Inflows to and outflows from the modelled system are described by specifying the flows (or water levels) and concentrations at the boundary.

Many alternatives to models based on the St Venant equations exist for simulating flow routing in streams. As mentioned briefly in §3.3.2, some flow routing models are based on the continuity equation and simplifications of the dynamic equation. It was noted in §3.3.2 that the bulk of a flood wave moves as a kinematic wave and that storage processes therefore dominate the routing. A separate class of flow routing models use the storage concept as their basis. These are often referred to as hydrologic routing models, perhaps the best known of which is the Muskingum method (Henderson, 1966).

The Muskingum method is based on a linear relationship between the storage in a reach V, and the inflow to the reach I and outflow from the reach O.

$$V = K[O + X(I-O)] \tag{3.11}$$

In Equation 3.11 X is a weighting parameter. The continuity equation for the reach is:

$$\frac{dV}{dt} = I - O \, . \tag{3.12}$$

Equations 3.9 and 3.10 are solved using an explicit finite difference scheme. Details of the finite difference scheme and application of the model may be found in Cunge (1969) and Weinmann and Laurenson (1979); however the following features of the model are enlightening.

Equation 3.11 implies a unique relationship between the flow and stage and is therefore equivalent to a kinematic wave model, one property of which is that the peak discharge remains constant as the flood wave progresses downstream (Cunge, 1969). However the numerical scheme used in the Muskingum method introduces an error which results in some numerical diffusion and thus attenuation of the peak discharge. Cunge (1969) showed that the numerical scheme used in the Muskingum method actually provides a solution to the diffusive wave equations (St Venant equations with the acceleration terms deleted) provided the weighting parameter is chosen correctly. Models which utilise this characteristic are referred to as Muskingum-Cunge models. Models based on storage methods such as the Muskingum and Muskingum-Cunge methods have the advantage of being computationally economical and relatively simple (Garbretch and Brunner, 1991) and many such models exist.

Before discussing simplified solute routing models it is necessary to consider numerical errors which can occur in solutions of the Advection-Dispersion equation. It is noted that similar errors can occur when solving the St Venant equations; however they are generally less severe (Sobey, 1984). Such errors can lead to serious numerical diffusion and oscillations in the solution and are linked to the discrete approximation of the advective term ($U \frac{\partial C}{\partial x}$ in Equation 3.9) (Sobey, 1984). The source of these errors can be appreciated by considering a Taylor series expansion for the concentration:

$$C(x+\Delta x, t) = C(x, t) + \frac{\partial C}{\partial x}\Delta x + \frac{\partial^2 C}{\partial x^2}\frac{\Delta x^2}{2} + \frac{\partial^3 C}{\partial x^3}\frac{\Delta x^3}{3!} + \ldots \qquad (3.13)$$

Equation 3.13 can be rearranged to give:

$$\frac{\partial C}{\partial x} = \frac{C(x+\Delta x, t) - C(x, t)}{\Delta x} - \frac{\partial^2 C}{\partial x^2}\frac{\Delta x}{2} - \frac{\partial^3 C}{\partial x}\frac{\Delta x^2}{3!} - \ldots \qquad (3.14)$$

Now if $\frac{\partial C}{\partial x}$ is approximated with $\frac{C(x+\Delta x, t) - C(x, t)}{\Delta x}$, the higher order terms in Equation 3.14 are ignored or truncated. It is this truncation which can lead to the numerical errors in the solution (Cunge et al., 1980).

The effects of numerical errors can be envisaged by considering the solution to the Advection-Dispersion equation (ie. the curve representing the distribution of solute along the river channel) as a Fourier series. Oscillations can be caused by amplification of short wavelengths components of the solution. Numerical dispersion, which occurs when different Fourier components move at different speed through the solution, also leads to oscillations when the low and high frequency components become separated. Numerical diffusion is caused by the damping of high frequency components of the solution. This reduces spatial gradients and smears the solute out. Fourier techniques can be used to quantify the above errors. The errors described above are usually most severe when concentration variations occur over short distances compared to the spacing of computational grid points (Sobey, 1984). Therefore the problems are worst near steep spatial gradients.

An alternative and more intuitive explanation of numerical diffusion is given in Figure 3.2 following Fischer et al. (1979). Solute is advected from j to a point between j+1 and j+2 during one time-step Δt. However, because mass is concentrated at the grid points, the solute is then proportioned between j+1 and j+2 and is consequently spread or dispersed along the channel.

Simple models can be used to model the transport of solutes in streams; however severe numerical errors can occur in some cases. One approach is to use a series of cells for which simple mass balance calculations are performed and between which solutes are transferred with the flow. Such models can lead to large numerical errors because the solute is mixed instantly across the cell and are generally not suitable for modelling streams with large concentration gradients.

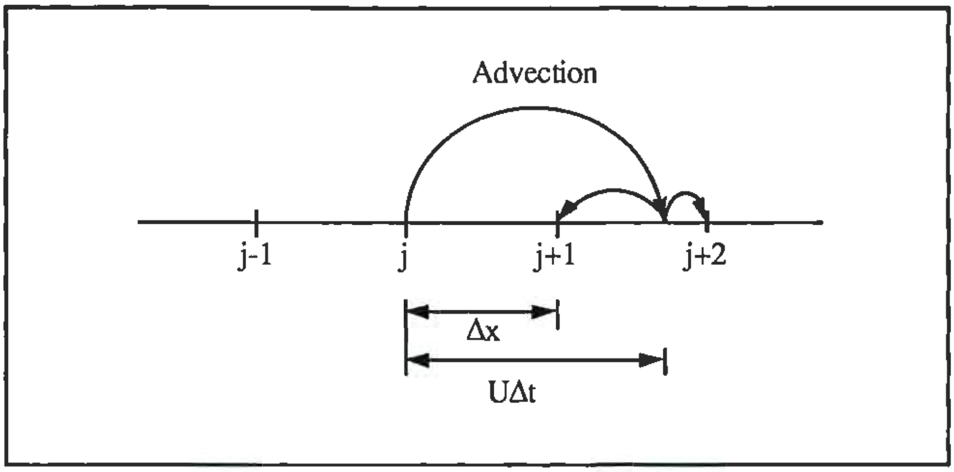


Figure 3.2: Numerical dispersion in a simple finite difference solute transport model. With $U\Delta t/\Delta x = 1^2/3$, the mass originating at point j is proportioned $^2/3$ to point j+2 and $^1/3$ to point j+1. After Fischer et al. (1979).

It was noted above that the discretisation of the advection term is a major cause of the errors in solutions of the Advection-Dispersion equation. These can be avoided by the simple expedient of using a Lagrangian approach in which the computational grid moves with the flow. This effectively eliminates the advective term from the equation consequently removes the numerical errors (Sobey, 1984; Fischer et al., 1979). One simple model which uses this approach was proposed by Fischer (1972). Packets of fluid are established which are moved up and down the channel on the basis of the volume of water stored in the channel. Dispersion is included through an exchange flow between neighbouring packets and results may be extracted at any point by interpolation; however interpolated concentrations are not used in subsequent calculations and thus do not lead to any numerical error.

3.5.1 MODEL SELECTION.

When selecting the model for this study the following factors were considered important. The role of the model was seen as providing an analytical tool for aiding the understanding of the behaviour of salinity in the Wimmera River and as providing a basis for future management planning. That is, the model itself was not a subject of the research. It was also felt that using a currently available model was appropriate as time spent writing computer programs would be minimised.

Given the role of the model, ease of use was seen as a major advantage. Furthermore the model needed to be capable of simulating flow events which were expected to have a major impact on water quality and of long term simulations because extended low flow periods (that can last multiple years) were expected to be critical from a water quality perspective. Finally the ability to alter the model algorithms was required so that an appropriate description of saline pools could be incorporated once it had been developed.

Four models were considered in detail: HSPF (Johanson et al., 1984), HEC5Q (HEC, 1975), WASP4 (Ambrose et al., 1988) and MIKE 11 (DHI, 1992a; 1992b). HSPF and HEC-5Q use storage routing for flow calculations while WASP4 and MIKE 11 solve the St Venant equations in rectangular and irregular channels respectively. HSPF uses a cells in series system and mass balance approach for water quality simulations. Solute transport is simulated using a finite difference scheme in HEC-5Q. Both HEC-5Q and HSPF use one-dimensional solute transport algorithms. WASP4 has a quasi three dimensional solute modelling system in which the water body is divided into compartments and the transport is simulated using an explicit finite difference system. WASP4 is the only one of the four models to include details of the accuracy of the numerical scheme in the associated program documentation. Numerical dispersion can be large in some circumstances (Ambrose et al., 1988). MIKE 11 includes a one-dimensional finite difference advection-dispersion model.

Program source code was available for all models except MIKE 11. However the source codes for HSPF and HEC-5Q are 75 000 and 120 000 lines long respectively. Codner (1991) suggests that even though HSPF is well structured and written its sheer size makes it relatively difficult to understand and modify. The HEC-5Q program source code is larger and less well documented than that for HSPF. The ease with which the WASP4 program could be modified was unclear. While the program source code for MIKE 11 was unavailable; a facility which allows addition of subroutines exists thus allowing incorporation of saline pools. It was believed that, of the four programs, it would probably be easiest to incorporate site specific changes to MIKE 11.

One significant disadvantage with both MIKE 11 and WASP4 was that the solute transport simulations are de-coupled from the flow simulations. This makes simulation of flow management strategies which utilise real time water quality information difficult. Obviously this limits the utility of a modelling system for exploring management options in streams such as the Wimmera River where water distribution can be controlled and where significant water quality problems exist.

A major advantage of MIKE 11 was that it includes a menu-driven interactive system for constructing the model and associated time-series data files which are properly integrated within the modelling system. The other models did not include such a facility.

Of the four models, all except HEC-5Q would have been capable of modelling the important features of flows in the Wimmera River. The major limitation with HEC-5Q was that it only supported time-steps of 1 day and 1 month. MIKE 11 includes a more detailed flow description than WASP4 although they are both based on the St Venant equations and MIKE 11 was apparently easier to use. Furthermore MIKE 11 was believed to be more easily modified and easier to use than HSPF. MIKE 11 was therefore chosen as the model to be used in this study.

3.6 MIKE 11.

MIKE 11 (DHI, 1992a; 1992b) was developed at the Danish Hydraulic Institute (DHI) for simulating flows, sediment transport and water quality in streams, estuaries and other similar water bodies (DHI, 1992b). MIKE 11 is available in a modular format. The modules used in this study were the Hydrodynamics and Advection-Dispersion modules. Other modules include water quality, sediment transport, dam break and a rainfall-runoff module. The following discussion is based on the MIKE 11 Technical Reference Manual (DHI, 1992b). Only those aspects of MIKE 11 of relevance to this study are described.

3.6.1 GENERAL DESCRIPTION.

MIKE 11 simulates the flow of water and transport of solute in irregular open channels using the St Venant and Advection-Dispersion equations which are solved using finite difference techniques. A link-node system is used to describe the river channels. A graphical user interface is used to create model files, to control simulations and to present the results of simulations.

3.6.2 SOLUTION OF THE ST VENANT EQUATIONS.

MIKE 11 solves the following equations for unsteady flow in an irregular channel:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\alpha \frac{Q^2}{A}\right) + gA\frac{\partial H}{\partial x} + \frac{gn^2 Q|Q|}{A^2 R^{4/3}} = 0 \qquad (3.15)$$

$$\frac{\partial Q}{\partial x} + B_{st}\frac{\partial H}{\partial t} = q. \qquad (3.16)$$

In Equations 3.15 and 3.16 Q is the discharge, H is the elevation of the water surface relative to a fixed datum, A is the cross-sectional area, R is the hydraulic

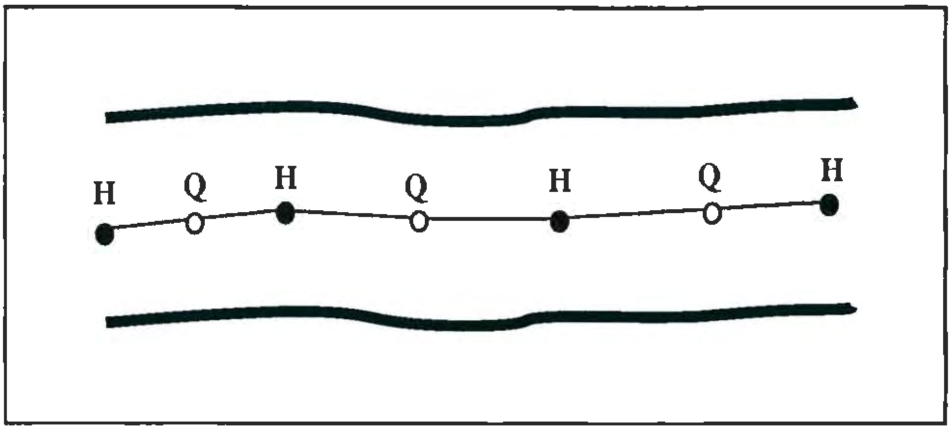


Figure 3.3: The computational grid used by MIKE 11 to solve the St Venant equations.

radius, B_{st} is the storage width, q is the lateral inflow per unit length, n is the resistance parameter in Manning's equation, α(=1 in this application) is a momentum distribution coefficient and x and t represent the spatial and temporal dimensions respectively. MIKE 11 is capable of solving Equations 3.15 and 3.16 or simplifications thereof. In this research the complete equations were solved for the numerical experiments conducted in Chapter 5 and the diffusive wave equations were solved when modelling the Wimmera River itself. The diffusive wave equations may be obtained from the complete equations by neglecting the first two terms in Equation 3.15.

In MIKE 11 solution of the unsteady flow equations is achieved using an implicit scheme based on that developed by Abbott and Ionescu (1967). The dynamic and continuity equations are solved using a computational grid consisting of alternating H-points and Q-points (Figure 3.3). The water level is computed at H-points or nodes and the discharge is computed at Q-points or links. That is, the dynamic and continuity equations are solved at different but intermeshing locations in space. Channel reaches always begin and end at H-points. The appropriate continuity equation is solved at confluences and bifurcations. Details of the solution scheme for both the complete equations and dynamic wave equations are provided in Appendix 1.

3.6.2.1 Hydraulic Structures.

MIKE 11 incorporates structures by replacing the dynamic equation at a specific Q-point with an appropriate structure equation. The solution then proceeds as normal. While MIKE 11 is capable of simulating a number of different hydraulic structures, the model of the Wimmera River has only utilised the broad-crested weir description. Both drowned and undrowned flow over a broad-crested weir can be described and MIKE 11 automatically chooses between the two. The description of broad-crested weirs used in MIKE 11 is explained in detail in Appendix 1.

3.6.2.2 User-Defined Structures.

User-defined structures consist of purpose-written subroutines that describe special features of the river system that are not already described by MIKE 11. It is possible to access information and to alter the behaviour at any point in the computational domain. Information can be supplied to the program from custom designed data files. While it is possible to alter the continuity description, the subroutines have only been used to control flows within the model of the Wimmera River and to obtain summary information. When specifying discharge at a particular location the subroutine replaces the dynamic equation at that Q-point. The subroutines operate by modifying the coefficients in the solution scheme (see Appendix 1, Equations A1.6 and A1.9).

3.6.2.3 Low Flow Computations.

During low flows the water level tends to fall below the bed in some cross-sections. This introduces numerical difficulties which can prohibit the continuation of computations unless special precautions are taken. MIKE 11 deals with the low flow problem by placing a slot in the cross-section which begins 0.1 m above the invert and continues to 0.5 m below the invert. Computations are unaffected by the slot when the water level is above the top of the slot. If the water levels falls below the bottom of the slot MIKE 11 adds additional water. The solution scheme coefficients are also altered when the water surface enters the slot. Computations during periods when the water surface is in the slot should not be given too much weight.

The above algorithm caused significant difficulties when modelling the Wimmera River since a large amount of water was being added artificially during low flow periods. This was overcome by specifying a series of broad-crested weirs in the river. Chapter 6 provides more details of the difficulties and solutions to low flow modelling difficulties in the Wimmera River.

3.6.2.4 Boundary Conditions.

When modelling streams using MIKE 11, upstream and downstream boundary conditions and lateral inflows must be specified. The upstream boundary condition and all lateral inflows are typically (and were in this study) specified using discharge time-series. Time-series may be specified with arbitrary time increments. MIKE 11 linearly interpolates the time-series to obtain the discharge at each simulation time-step. Downstream boundary conditions in streams can be specified using a rating curve. It should be noted that a rating curve is not a true dynamic boundary condition since the real relationship between flow and water level is not unique (Zoppou and O'Neill, 1981).

3.6.3 SOLUTION OF THE ADVECTION-DISPERSION EQUATION.

MIKE 11 simulates solute transport by solving the following Advection-Dispersion.

$$\frac{\partial AC}{\partial t} + \frac{\partial QC}{\partial x} - \frac{\partial}{\partial x}\left(AD\frac{\partial C}{\partial x}\right) = C_s.q \ . \qquad (3.17)$$

In Equation 3.17 A is the cross-sectional area, C is the concentration, D is the dispersion coefficient, C_s is the concentration of any lateral inflow and q is the lateral inflow per unit length. Equation 3.17 is solved using an implicit third order correct finite difference scheme. The concentration is calculated at all (H-points and Q-points) computational points. Details of the solution are provided in Appendix 1. Results from the flow simulations are used to obtain the discharge and cross-sectional area.

3.6.3.1 User-Defined Structures.

It is possible to write subroutines that modify the solution of the Advection-Dispersion equation. These subroutines operate in the same way as those described in §3.6.2.2.

3.6.3.2 Dispersion Coefficient and Boundary Conditions.

The dispersion coefficient can be specified such that it is related to velocity using a power curve and this relationship can vary spatially. Solution of the Advection-Dispersion equation requires specification of the discharge and concentration of all inflows. It is noted that lateral inflows may be included in advection-dispersion simulations even if they haven't been included in the flow simulations. Lateral inflows should not be omitted from the flow simulations unless their impact on the water balance is insignificant.

3.7 DATA REQUIREMENTS.

The data required to model the Wimmera River can be divided into three categories: boundary condition data, calibration and verification data, and cross-section data. Boundary condition requirements were mentioned above. To specify the model boundary conditions, time-series of discharge and concentration are required for all inflows and stage-discharge relationships are required for outflows. Development of a consistent set of time-series boundary condition data is described in detail in Chapter 4.

3.7.1 CALIBRATION AND TESTING.

Calibration and testing is discussed in greater detail in Chapter 6; however data requirements are summarised briefly below. Calibration and testing of the model require observations of the hydraulic and hydrologic response that can be compared to the model simulations. Useful data include the discharge as a function of time, the stage as a function of time, the stage-discharge relationship, water surface profiles and the salinity as a function of time. These data are required at several locations along the river.

3.7.2 RIVER GEOMETRY.

The river geometry is specified in MIKE 11 by specifying the channel network and a series of cross-sections in each channel. The channel network can be specified using aerial photographs and topographic maps. A series of channel cross-sections which can be related to the Australian Height Datum (AHD) and river chainage are also required. Chapter 5 discusses specification of the stream geometry in detail.

3.8 SUMMARY.

Fundamental principles of open channel flow, including the concepts of subcritical and supercritical flow and hydraulic control, were discussed at the beginning of this chapter. The St Venant equations for gradually varied flow in open channels were then introduced and flow resistance and flow routing discussed. The movement of flood waves is dominated by storage of water in and adjacent to the stream channel. The transport of solute in open channel flows can be described using the Advection-Dispersion equation. Shear flow dispersion was discussed in the context of natural streams. Dispersion is most significant where large spatial gradients exist.

Stream modelling was then summarised. Stream models can be based on the St Venant and Advection-Dispersion equations, on simplifications of those equations or on storage and continuity principles. The Muskingum-Cunge method for flow routing and a simple Lagrangian solute transport model were introduced. Four different stream models were described briefly and the reasons for choosing MIKE 11 to model the Wimmera River were summarised. Finally MIKE 11 was described in some detail and the data requirements for the modelling component of this study were identified.

CHAPTER 4 - DEVELOPMENT OF A TIME-SERIES DATA BASE FOR MODELLING THE WIMMERA RIVER.

4.0 INTRODUCTION.

Modelling of flow and solute transport in streams using models such as MIKE 11 was discussed in Chapter 3. To apply MIKE 11 to the Wimmera River a set of boundary condition data for each inflow and outflow must be developed. This chapter discusses the development of a data set containing time-series of flow and concentration which was used to provide inflow boundary conditions in the MIKE 11 model of the Wimmera River. Data points in the series must be frequent enough to define temporal variations in flow and solute concentration for each inflow to the Wimmera River included in the model. Where these data are unavailable or of inadequate frequency then additional data additional data must be estimated.

The available boundary condition data for the Wimmera River are described and assessed in this chapter. Various methods for estimating flow and salinity data are examined and the most appropriate methods are selected. Details of the methods used are provided. Because infilling of solute concentration data is required for all inflows rather than a small proportion of the inflow as is the case for discharge, more emphasis has been placed on the techniques used to infill solute concentration data. Data sets used to describe the interaction between the groundwater and the Wimmera River are also developed.

4.1 AVAILABLE BOUNDARY CONDITION DATA.

Figure 4.1 is a schematic of the Wimmera system that shows the location of surface water inflows to the Wimmera River and relevant surface water monitoring stations. The stations are those that are closest to the model boundary on the respective tributaries. The characteristics of the different types of data have been described in Chapter 2.5.

There are two problems with the available data set. Firstly, a number of ungauged catchments flow into the reaches of the Wimmera River that are being modelled and secondly, the salinity data that are available are usually not frequent enough to define the temporal variation of the salinity of flows entering the river. The following discussion outlines methods of estimating data where the data are inadequate. Although some additional data collection was possible it was limited by financial constraints.

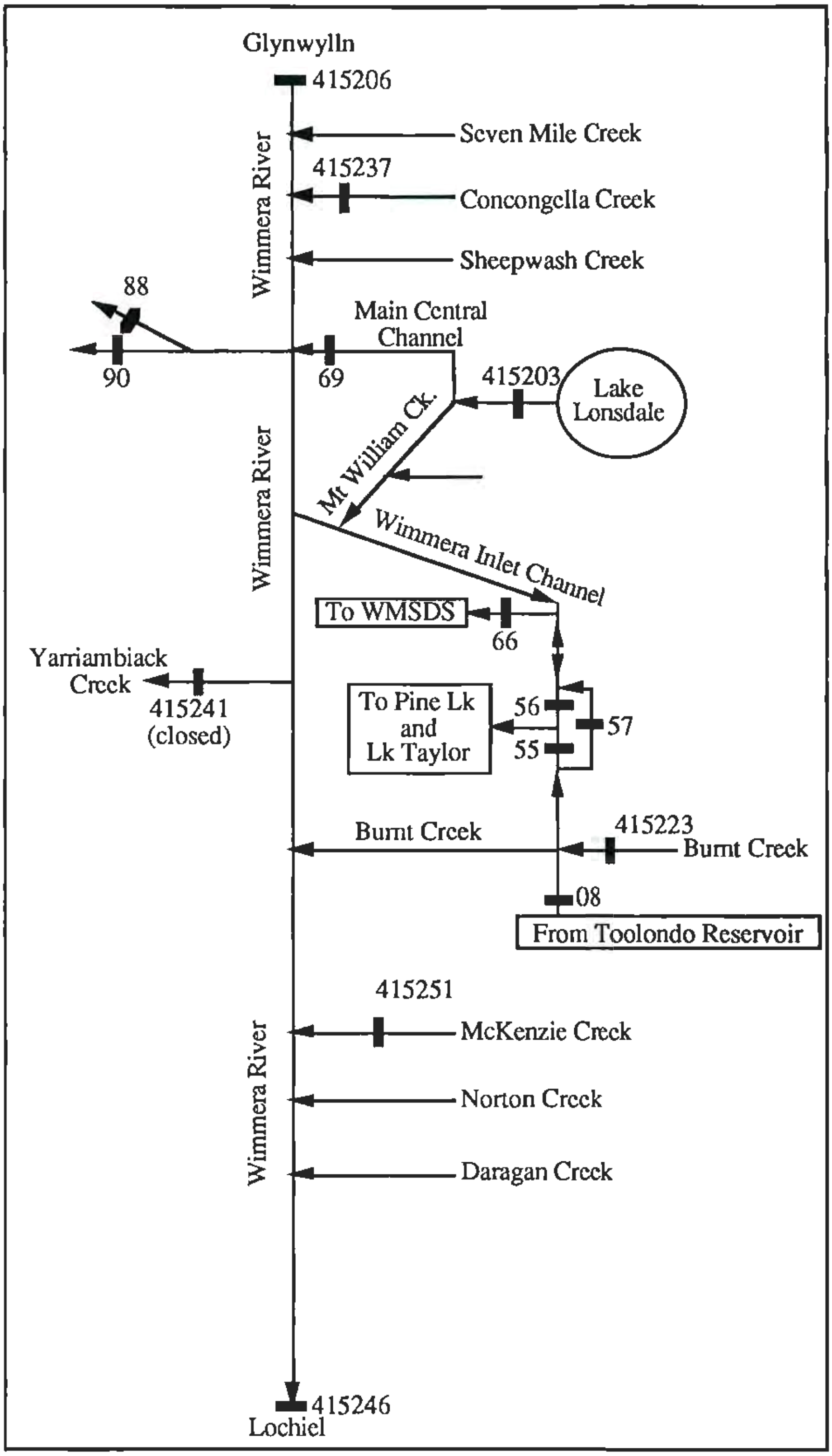


Figure 4.1: Inflows and outflows of surface water to and from the Wimmera River between Glynwylln and Lochiel. Relevant flow monitoring stations are shown. Those with a 415 prefix are hydrographic gauging stations operated by the Rural Water Corporation of Victoria and those with a two-digit identification number are monitoring locations within the Wimmera Mallee Stock and Domestic Water Supply System.

Water and solute balance calculations have been performed for the streams and channels in the Wimmera River Catchment for the thirteen year period between 1976 and 1988 (Hooke, 1991). Total discharges were calculated using a combination of gauging and volume balancing while salt loads were calculated using flow-weighted mean conductivities and mass balancing. These calculations form the basis of the following discussion. The above report indicates that the flows from the ungauged parts of the catchment are approximately 31% of the flow from the catchment at its outflow (Lochiel, 415246) or 15% of the total flow generated within the catchment. The difference in these values is due to 48% of the catchment yield being harvested and 3% of the catchment yield passing down Yarriambiack Creek which is a distributary stream of the Wimmera River.

Calculated salt load balances indicate a significantly different situation with respect to salt loads associated with ungauged flows. Ungauged flows include direct groundwater intrusion to the river which, although not a large water source, are a significant salt source. Salt loads in ungauged flows are 50% of the total salt load generated within the catchment and 137% of the salt load flowing out of the catchment at the major natural outflow (Lochiel). There is greater uncertainty associated with the calculated salt load figures since they are based on discontinuous, usually monthly or irregular, conductivity samples. Further uncertainty arises from the fact that the gauging station at Lochiel (415246) is a relatively new station which has only operated since February 1988 and thus has a very short record in the study period. While these uncertainties exist, the significant difference in the above values is expected due to the effects of water harvesting.

Hooke (1991) estimates that groundwater flows account for 28% of the total ungauged salt load; however this may be an underestimate. The salt load due to groundwater inflows is only calculated downstream of Horsham. However it is likely that base-flows entering the river upstream of Horsham also lead to a significant increase in salt load. There are two sources of evidence for this.

Firstly, saline groundwater discharge to the river in the Roseneath area upstream of Horsham, has been documented by Anderson and Morison (1989c) and Strudwick (DCE, Per. Com.). Secondly, by far the most significant contribution to salt loads from ungauged tributaries upstream of Horsham enters the river between Glynwylln and Glenorchy (40% of the ungauged salt load). Over this reach it is estimated that 18 400 t/a of salt enters in 11 000 Ml/a of water. This is equivalent to a flow weighted mean salinity of 2 800 EC units. This salinity is an order of magnitude higher than any other tributaries in the catchment and indicates that a significant proportion of this salt load is likely to be entering in groundwater inflows rather than tributary inflows.

It is clear that a significant amount of data estimation or infilling was required. Before data were infilled, the time increment at which the data were to be infilled had to be selected. Major considerations when selecting the time increment include, the time-step used in modelling, the time-scale over which hydrologic fluxes change in the catchment and the time-scale at which estimates can be made with reasonable confidence. Most major water quality changes in the Wimmera are associated with discharge events and typical hydrograph rise times are approximately 12 hours for the Wimmera River at Glynwylln. To define the rising limb of a hydrograph data would therefore be required at least every 6 hours. Given that the major inflow to the reach of the Wimmera River being modelled occurs at Glynwylln and that there is continuous flow data available, estimates of salinity were made at 6 hourly intervals.

At many other locations both flow and salinity required estimation. It was believed that estimates of average flows or salinities at time-scales less than one day would be unrealistic yet significant changes in hydrologic fluxes in the tributaries of the Wimmera River occur in one day. Therefore mean daily surface water inflows and salinities were estimated for these sites. MIKE 11 treated these flows as instantaneous flows and interpolated between the values, which is somewhat inconsistent with the characteristics of the estimated data. However it is believed that the errors in estimated daily fluxes are significantly greater than any error introduced by treating mean flows as instantaneous flows and, given the simplicity of the adopted approach, it is preferred over any attempt to distribute tributary flows within each day.

4.2 FLOWS FROM UNGAUGED CATCHMENTS.

There are several possible methods of estimating flows for ungauged catchments. The various methods generally rely on adjusting flows from a nearby gauging station or stations (eg. Fuller, 1978) or using a simple rainfall-runoff model (Laurenson and Jones, 1968). Generally there is a trade-off between simplicity and accuracy for the various methods (Laurenson and Jones, 1968). Rainfall-runoff models tend to be more complex than methods relying on nearby gauges.

It was believed that the additional effort required to apply a rainfall-runoff model to a series of ungauged catchments would not be justified by the likely improvement in performance of the river model. Furthermore it was not possible to test the predictions of a rainfall-runoff model applied to specific ungauged tributaries of the Wimmera River. Therefore flows from nearby gauges, with a suitable adjustment, were used in preference to a rainfall-runoff model to estimate flows from ungauged tributaries. To some extent this approach was vindicated by subsequent simulations with the MIKE 11 model which generally predicted flow relatively well (Chapter 6).

Of those methods relying on nearby gauges, the simplest possible method is to simply scale the hydrograph at a nearby gauging station by the ratio of the catchment areas. This assumes a linear relationship between catchment area and runoff and that the two catchments have similar flow sequences (ie hydrograph shape and timing). A better approach would be to scale the hydrograph by the ratio of the mean annual discharges of the two catchments. This removes the assumption of linearity between catchment area and flow but retains the assumption that the two catchments have similar flow sequences. One problem with this approach is that the ratio of mean annual flows is required, but the mean annual flow for the ungauged catchment is unavailable.

If it is assumed that scaling a known hydrograph by the ratio of mean annual flows for the two catchments is adequate, the problem of generating flow sequences is reduced to one of predicting the mean annual flow for the ungauged catchment. A number of studies have been undertaken which have used catchment area when estimating mean annual flow. The following is a brief discussion of some that have been reported in the literature. It is not and not meant to be an extensive review of the studies.

McMahon (1976) studied 156 catchments over Australia and concluded that the ratio of mean annual flows for nearby catchments could be predicted using:

$$\frac{Q_1}{Q_2} = \left(\frac{A_1}{A_2}\right)^{0.65} \tag{4.1}$$

where Q_1 and Q_2 are the mean annual discharges and A_1 and A_2 are the catchment areas. Other researchers have used similar approaches including Torelli and Tomasi (1977) who related mean annual flows in the Emilia-Romagna region of Italy to catchment areas. Fuller (1978) predicted the relationship between an ungauged site and several nearby gauges in Ontario, Canada by combining the flows from the gauged catchments in a linear equation in which the coefficients were predicted from physiographic variables. The variables included drainage area, slope, mean monthly precipitation, drainage density and mean monthly temperature. Horn (1988) found that area, mean annual rainfall and percent

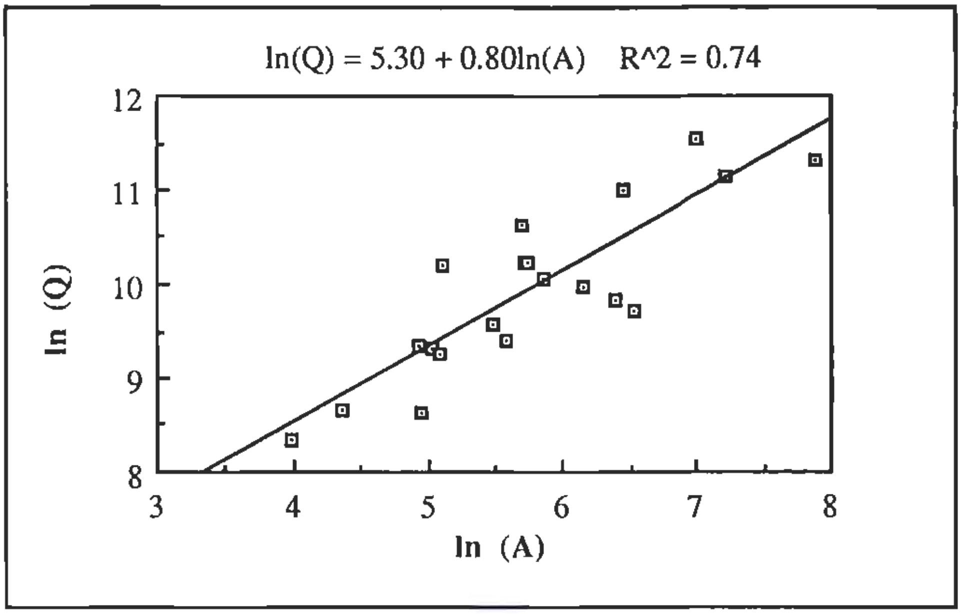


Figure 4.2: Relationship between catchment area (A) and mean annual discharge (Q) for streams in the Wimmera River and neighbouring catchments.

forested were significant when predicting mean annual flows in Idaho, USA. Gan et al. (1990) found that area and mean annual rainfall were important predictors of mean annual flows in south east Victoria, Australia. An innovative study by Nathan (1990) utilised Andrew's curves to group catchments on the basis of physiographic variables in south east Australia. Predictive relationships for mean annual flows and other flow properties were then derived for each group. Variables that were most often significant in prediction were catchment area and rainfall, however these were not significant in all groups of catchments.

To estimate flows from ungauged catchments for use in the MIKE 11 model, hydrographs were scaled by the ratio of mean annual flows for the catchments. An approach based on McMahon (1976) was used to estimate mean annual tributary flows. Equation 4.2, which is based on a regression of mean annual flow against catchment area for uncontrolled streams in the Wimmera and nearby catchments, was used rather than Equation 4.1 which was developed for catchments Australia wide.

$$\frac{Q_1}{Q_2} = \left(\frac{A_1}{A_2}\right)^{0.8} \tag{4.2}$$

Figure 4.2 shows the relationship between mean annual discharge and catchment area for the catchments used in the regression.

The major ungauged tributaries contributing flow to the Wimmera River between Glynwylln (415206) and Lochiel (415246) are the lower Concongella Creek and surrounding minor tributaries, the lower Mt William Creek and surrounding minor tributaries, Burnt Creek, Norton Creek, Daragan Creek and the McKenzie River. The McKenzie River has a new gauging station with a short record available. Flows in the McKenzie River were infilled using a regression relationship with flow in Burnt Creek which is a neighbouring stream. Mean annual flows for other ungauged catchments and sub-catchments were estimated using Equation 4.2.

It is noted that Equation 4.2 implies that a reduction in flow per unit area occurs as catchment area increases. This is partly due to the tendency for small catchments to be located high on the slopes of the ranges forming the catchment watershed and for larger catchments to incorporate more land with a lower elevation and less rainfall and runoff. Therefore when applying Equation 4.2 to an ungauged area located at the bottom of a catchment the following approach was adopted. The mean annual flows for the whole catchment and for the upper gauged portion of the catchment were estimated with Equation 4.2 and the difference between these two mean annual flows was used as an estimate of the mean annual flow from the ungauged portion of the catchment. The total flow was then estimated as the sum of flows from the gauged and ungauged parts of the catchment. Flows from all ungauged tributaries, with the exception of McKenzie Creek, were estimated by scaling the hydrograph for Concongella Creek at Stawell (415237). This site was chosen because it is the closest gauged tributary to the sites where flow data must be infilled that is unaffected by regulation. Details of the estimation of flows for specific streams are provided in §4.7.

4.3 WATER QUALITY BEHAVIOUR IN UNREGULATED STREAMS.

The solute load at a catchment outlet reflects catchment-wide processes. It is also sensitive to changes in catchment hydrology and can provide a measure of human impact on a catchment (Walling, 1984). Sources of solutes carried by streams include atmospheric deposition, rock and soil weathering (Walling, 1984, Macumber, 1991), pollution by man (Walling, 1984) and connate salts (Macumber, 1991). A number of factors have a strong influence on the average concentration of stream solutes including: the relative importance of atmospheric sources and rock and soil sources; evaporation; and catchment physiography including relief and geology (Walling, 1984).

Solute concentrations vary significantly over time. The temporal behaviour of solute concentration is influenced by the flow rate (Ledbetter and Gloyna, 1964; O'Connor, 1976; Foster, 1978; Walling, 1984; Sivakumar et al., 1988), the balance between flow components (O'Connor, 1976; Hart et al., 1964; Pionke et al., 1972; Walling, 1984), antecedent conditions (Ledbetter and Gloyna, 1964; Foster, 1978), seasonal effects (Foster, 1978; Walling, 1984), catchment exhaustion (Walling, 1984) and channel routing effects (Glover and Johnson, 1974). Of these the most important is generally the stream discharge rate.

Water quality varies with stream discharge rate because the flow rate and concentration of water sources vary. During low flow the water in a stream is predominantly base-flow from groundwater discharging to the stream. When a storm occurs in the catchment, surface runoff is generated which has a different, typically lower, solute concentration. This surface runoff mixes with and dilutes the base-flow. This process was described by Hem (1948) in the Rio Grande and Pecos rivers, New Mexico, USA and has been recognised by many other authors (Durum, 1953; Johnson et al., 1969; Walling and Foster, 1975; Foster, 1978; Walling, 1984; Sivakumar et al., 1988; others) since. If it is assumed that a given discharge is the result of mixing of water from a number of sources and that the flow and concentration of each of these sources is always the same for that discharge then all the variation in solute concentration can be explained by the stream discharge alone. Predicting solute concentrations would simply involve sampling the stream to define the relationship between flow and concentration and using a rating curve to predict concentrations.

While the dilution process is important there are other processes which influence the solute concentration in a stream. For a given stream discharge the proportion of base-flow can vary depending on antecedent conditions (Hart et al., 1964; Pionke et al., 1972, O'Connor, 1976). Pionke et al (1972) found that the ratio of surface-flow to base-flow was especially variable in ephemeral streams in the southern Great Plains of the United States. Furthermore, since the majority of the salt load in these streams is associated with the base-flow component the salinity variation due to changes in the surface-flow to base-flow ratio was significant.

The surface-flow to base-flow ratio can also vary between the rising and falling limbs of a hydrograph. During a storm event base-flow generally increases due to recharge of the contributing aquifers which means that at the same stream discharge the base-flow component will be greater on the falling limb. Provided the surface flow has a lower concentration than the base-flow this leads to an anti-clockwise hysteresis on a flow versus concentration plot (O'Connor, 1976; Walling, 1984).

The concentration of solutes in the runoff components is also variable. Seasonal effects have often been observed in the behaviour of stream solute concentrations. Roberts et al. (1984), Hill (1986) and Roberts (1987) have described seasonal effects in which the concentration of nitrate in streams decreased over the winter period. Similar effects have been described by Cornish (1982) for specific conductivity in streams draining forested catchments in New South Wales. The reduction in conductivity has been interpreted as evidence of reducing solute availability as they are leached from the soils during the wet part of the year (Walling, 1984).

A similar process operating at a shorter time-scale has also been described (Walling and Foster, 1975; Foster, 1978; Cornish, 1982; Stokes and Loh, 1982; Walling, 1984). When a storm occurs, solutes that have accumulated on the catchment surface during the preceding dry period are dissolved and washed from the catchment at the start of the storm. As the storm progresses the amount of solute available in the catchment decreases, resulting in decreasing concentrations as the storm progresses. The effect of this is to produce a clockwise hysteresis in the concentration versus flow plot and the process is referred to as catchment exhaustion.

Spatial variation in runoff generating areas, especially in larger catchments, also contributes to the variability of stream solute concentrations. Hem (1948) describes occasions in the Rio Grande when the electrical conductivity has increased during flood periods and others when the electrical conductivity has decreased. The reason for this apparently anomalous behaviour was that when the specific conductance increased during a flood the source of the surface runoff was a catchment containing soluble rocks. This led to the surface runoff having a high electrical conductivity. On other occasions the source of the surface runoff was a tributary in which the electrical conductivity of the surface runoff was low.

Glover and Johnson (1974) examined another cause of variation in water quality in rivers. When a flood wave moves down a river its celerity is greater than the advective velocity of the flow. A lag between the rise in discharge and change in water quality develops which is especially noticeable in larger rivers and can cause a clockwise hysteresis in the discharge versus concentration plot.

The above summary only considers major causes of variation in solute concentrations. With many processes operating in a catchment there is obviously the potential for significant variations in solute concentrations in streams. While there are a significant number of processes they can be grouped into two groups.

The first of these includes the dilution effect and processes that influence the base-flow to surface-flow ratio and the spatial variation in runoff which are processes that cause a variation in the proportion of flow from each source. The second includes those processes such as short and long term variations in solute availability that result in changes in concentration of sources of flow.

4.4 MODELLING TEMPORAL VARIATION IN SOLUTE CONCENTRATION.

4.4.1 DILUTION MODELS.

Several different methods of modelling the temporal variation of solute concentration are available. The simplest is a rating curve relating concentration to flow. Such a method assumes that there is a unique, time-invariant relationship between concentration and flow (Rieger et al., 1982). This is a perfect dilution effect. Of course this assumption is never completely satisfied in a catchment; however relationships that explain 95% of concentration variation have been obtained using regressions of the form:

$$C = aQ^b \qquad (4.3),$$

where C is the solute concentration, Q is the stream discharge and a and b are constants. The relationship between concentration and flow is much more variable in other catchments (Walling, 1984). Seker (1991) has compared the load prediction ability of the log-linear model (4.3) with an exponential model (4.4) in several streams in the Goulburn River Catchment in north-central Victoria.

$$C = a\, e^{bQ} \qquad (4.4),$$

Daily conductivities were more accurately estimated by the log-linear model; however, the evidence relating to long term load prediction was inconclusive.

Attempts have been made to justify different forms of the rating curve using physical reasoning based on the storage concept for water and solute. Hall (1970) derived various forms of rating curve by treating the catchment as a perfectly mixed volume containing a constant solute load or solute mass and for which the hydrologic budget was governed by a storage discharge relationship of the form:

$$Q = kV^n \qquad (4.5),$$

where Q is the discharge, V is the storage volume and k and n are constants. Combining Equation 4.5 with the assumption of constant load, L=CV, leads to:

$$C = A_c Q^{-1/n} \qquad (4.6),$$

where A_c is a constant given by $A_c = Lk^{1/n}$ in which L is the constant load. Equation 4.6 is the log-linear model.

Hall (1970) derived several other models including one based on an alternative hydrologic budget governed by:

$$Q = k(V - V_0)^n, \qquad V > V_0 \qquad (4.7),$$

where V_0 is a cease to flow volume. This results in the following relationship:

$$C = \frac{L/V_0}{1 + B_c Q^{1/n}} \qquad (4.8),$$

in which B_c is a constant given by $B_c = k^{-1/n} V_0^{-1}$.

The reasoning behind a number of other relationships developed by Hall (1970) is unclear. Hall attempted to include the effect of an inflow of constant concentration to the mixing volume but at the same time retained the assumption of constant load. Of course the assumption of constant load implies that there is always a source of solute that exactly balances the solute removed. This means that Equations 4.6 and 4.8 rather than those derived by Hall are appropriate. An attempt was also made to consider the case where either the concentration or flow, or both, only varied a small amount. Hall set the time derivative (Equation 4.9) of the solute load to zero and approximated other terms, thus deriving three different models.

$$\frac{dL}{dt} = C\frac{\partial V}{\partial t} + V\frac{\partial C}{\partial t} = 0 \qquad (4.9)$$

Apparently the models were then applied to large ranges in concentration and discharge. The reason for doing this is unclear. Given the assumptions made, both models 4.6 and 4.8 are exact so there is no need to approximate them. The assumption that either flow or concentration is restricted to a small range simply implies that model 4.6 or 4.8 is restricted to a small portion of the curve described by those equations.

Hall (1971) tested the various models derived by Hall (1970) on field data and concluded there was little difference between the fitted models. Hall then suggests that relying solely on the fit of a regression to chose an appropriate model is inappropriate, especially if extrapolation to a greater discharge range is required.

Johnson et al. (1969) successfully used a similar approach to Hall (1970, 1971) when analysing data from the Hubbard Brook Experimental Forest Catchment in New Hampshire, USA. Their model was based on a two-part mixing volume. The first part was a constant volume with a constant concentration associated with it and the second part was a variable volume, also with a constant but different concentration associated with it. Discharge from the volume was assumed to obey the law $Q = V/A$, where V is the variable volume and A is a constant. This leads to a model of the form:

$$C = \frac{C_\delta}{1+\beta Q} + C_\alpha \qquad (4.10),$$

where $C_\delta = C_0 - C_\alpha$, C_0 is the concentration associated with the fixed volume, C_α is the concentration associated with the variable volume and β is given by A/V_0 in which V_0 is the fixed volume. This model incorporates a dilution effect resulting from mixing of two water sources.

Walling (1984) has suggested using a rating curve with an adjustment factor to predict concentrations. The adjustment factor is calculated so that the predicted and sample concentrations are equal at either end of the period between samples. A linear interpolation of the adjustment factor is used between the samples. Walling uses a log-linear rating curve which, with the above adjustment factors, results in concentration predictions based on Equation 4.11.

$$C = \left\{ \frac{C_1}{aQ_1^b} + \frac{t}{T} \left(\frac{C_2}{aQ_2^b} - \frac{C_1}{aQ_1^b} \right) \right\} aQ^b \qquad (4.11)$$

In Equation 4.11 C is the predicted concentration, C_1 and C_2 are the concentrations of the previous and next sample respectively, Q_1 and Q_2 are the instantaneous discharge at the corresponding sample times, T and t are the time intervals between C_1 and C_2, and C_1 and C respectively, Q is the instantaneous discharge at the time at which the concentration is predicted and a and b are constants. This method attempts to allow for temporal variation in the dilution characterised by a time-scale longer than the sampling interval. However it relies on the ability of the point samples in a data series to describe the longer term temporal variation in the rating relationship. This requires that variation with a time-scale shorter than the sampling period be significantly less than variation with a time-scale longer than the sampling period. Walling does not provide any case studies to establish the merits of this model. In the Wimmera where data are typically monthly this method could possibly be used to approximate seasonal effects; however the models predictive ability must be established first.

4.4.2 ERRORS ASSOCIATED WITH DILUTION MODELS.

4.4.2.1 Load Prediction Errors.

Foster (1980) studied errors in predictions of monthly and annual dissolved loads based on regression relationships for a 1.6 km^2 catchment in east Devon, England. The study period was from June 1974 to September 1977 and a water year from September to August was used. Continuous flow data and six hourly water samples were collected from which concentrations of TDS, K^+, Ca^{++}, Na^+, Mg^{++}, Cl^- and NO_3^- were determined. Foster (1980) fitted regression relationships to sixteen months of six hourly data and used these to predict solute loads for a forty month period. The actual load over the forty months was calculated by integrating the six hourly samples with the instantaneous discharge at the time of sampling. Three different forms of regression equation were used, the most complex of which was Equation 4.12 which was referred to as the cubic equation.

$$C = aQ^b + cQ^d + eQ^f \qquad (4.12)$$

The two simpler forms of equation omitted one and two of the discharge terms and were referred to as the quadratic and linear forms of the equation respectively. Foster concluded that adding terms did not necessarily improve the fit of the regression equation. However for K^+ and Cl^-, for which the linear or log-linear model was a poor fit, the coefficient of determination increased from 0.057 to 0.371 and 0.09 to 0.405 respectively when the additional terms were added. Interestingly, the ability of the rating curves to predict loads of these species was not necessarily improved and in some years the equations with higher coefficients of determination were significantly worse. Foster (1980) used a Kolmogorov-Smirnov test to test for differences in the error distributions of the different rating equations and found significant changes with the quadratic and cubic models for Ca^{++} and K^+ but for no other solute species. Errors in monthly dissolved load estimates of greater than ±50% occurred for all species with all models. With the exception of K^+ and NO_3^- the majority of annual loads calculated were within ±25% of the actual load.

The 1975-76 water year in Foster's (1980) study was a drought year. The regression relationships used to predict loads were fitted to data collected prior to 1975-76. The predictions for 1975-76 were the poorest of the study period which indicates that models relying on a relationship between concentration and flow may not perform well in times of hydrologic extremes. This suggests that the use of such models in catchments with highly variable hydrologic conditions may lead to larger errors in the prediction of solute loads, especially during periods of extreme hydrologic conditions.

4.4.2.2 Biased Concentration Estimates.

When standard regression (ordinary least squares) techniques are used one of the assumptions required is that the error terms have an independent normal distribution with zero mean and a standard deviation, σ, which is independent of the explanatory variables. If the regression analysis is performed in the log domain then the predicted variable needs to be converted back to the real domain. Generally this will lead to a bias because the error distribution will no longer be symmetric.

Ferguson (1986) argues that in the case of the log transfomation this bias leads to a systematic under estimation of the predicted variable because the residuals (after re-transformation) have a log-normal distribution. Using the expected value of the log-normal distribution and the standard deviation of residuals Ferguson suggests that multiplying the estimate of the predicted variable by $e^{2.65s^2}$, where s^2 is an unbiased estimator of the variance of the residuals, σ^2, will remove any bias. However, the standard deviation of predictions is different from the standard deviation of residuals and, unlike the standard deviation of residuals, the standard deviation of the predicted variable increases as the extremes of the sample set of explanatory variables, in this case flows, are approached. Cohn et al. (1989) have demonstrated that Ferguson's method can lead to large over-estimates of concentration where estimates are made from flows near or beyond the extremes of the sample data set. Cohn et al. (1989) suggest that a minimum variance unbiased estimator (MVUE) (Equations 4.13 and 4.14) originally proposed by Bradu and Mundlak (1970) should be used to predict concentrations when a log-log model is appropriate.

$$C_{MVUE} = aQ^b.g_m\left(\frac{m+1}{2m}\{(1-V_p)s^2\}\right) \qquad (4.13)$$

$$g_m(z) = \sum_{p=0}^{\infty} \frac{m^p(m+2p)}{m(m+2)\ \ldots\ (m+2p)}\left(\frac{m}{m+1}\right)\left(\frac{z^p}{p!}\right) \qquad (4.14)$$

In Equations 4.13 and 4.14 C_{MVUE} is the estimated concentration or salinity, a and b are the rating curve coefficient and power respectively, Q is the discharge, m is the number of degrees of freedom in the error distribution, and s is an unbiased estimate of the error standard deviation in the log domain. V_p is the variability of predictions in the log domain and is given by:

$$V_p = \left\{ \frac{1}{n} + (\ln Q^* - \overline{\ln Q})^2 \Big/ \sum_{i=1}^{n} (\ln Q_i - \overline{\ln Q})^2 \right\} \qquad (4.15)$$

In Equation 4.15 n is the number of data points used to develop the rating curve, $\overline{\ln Q}$ is the mean of the natural logarithms of the flows used to develop the rating curve and $\ln Q^*$ is the natural logarithm of the flow for which the concentration is being estimated. In cases where the log-linear model is inappropriate neither of these methods would be appropriate.

4.4.3 ANTECEDENT CONDITIONS.

To overcome the problems associated with poor concentration prediction by simple dilution models some authors have included other variables which, in general attempt to account for the different sources of flow and/or varying solute concentrations from specific sources. Ledbetter and Gloyna (1964) examined rivers in the south-west United States which are characterised by highly variable flows and cross-sections, high TDS concentrations and large evaporation.

While Ledbetter and Gloyna found that the log-linear model could explain the data they suggested treating the power, b, in Equation 4.3 as a function of a set of explanatory variables could help to account for other influences on stream water quality. A dependence on the flow rate was initially considered; however rather than including more processes, this simply implies a different functional form of the rating curve. Dependence of b on the flow and an antecedent condition was also considered. The antecedent condition included was the average daily rainfall for the previous thirty days. The inclusion of average rainfall over the previous thirty days could help to account implicitly for the effect of catchment exhaustion and the effect of rainfall on the ratio of surface to base-flow; however it does neither of these explicitly.

Although Ledbetter and Gloyna claim that their formulation is superior to the log-linear model they do not provide specific case studies to illustrate this and, as is apparent from the study of load prediction errors using rating curves, an increase in the amount of explained variance in the concentration flow data set does not necessarily mean that solute load predictions will improve.

Both Foster (1978) and Sivakumar et al. (1988) have considered variables other than instantaneous stream discharge in regression models. Foster (1978) performed a step-wise multiple regression analysis on quality data from the same East Devon catchment as Foster's (1980) study. The most significant variable for all species was the instantaneous discharge. Water temperature, soil moisture deficit, antecedent precipitation index, half hourly rainfall intensity and a sine wave with an annual cycle were significant for some species. The antecedent precipitation index was a weighted sum of previous daily precipitation. For total dissolved solids only the stream discharge was significant. Sivakumar et al. (1988) considered the hydrograph shape, number of previous dry days and the total runoff (excluding base-flow from the event) in addition to the instantaneous storm runoff as explanatory variables for total dissolved solids concentrations in a small coastal catchment 80 km south of Sydney, Australia. Again only the instantaneous flow was significant.

4.4.4 RATING CURVES AND RELATED REGRESSIONS.

The various different models reviewed above attempt to relate concentration to flow and in some cases antecedent conditions. The authors proposing the models suggest that each has its specific advantages. It has also been suggested that the coefficient of determination may not be the best statistic for model selection because it may not be sensitive enough (Hall, 1971) and the highest coefficient of determination is not necessarily associated with the most accurate load prediction (Foster, 1980). While there may be advantages associated with certain models it should be remembered that rating curve models fitted by regression are essentially empirical models intended to transform a flow into an estimate of concentration. As such the appropriate model is the one which provides the best predictions. If different models perform equally well then the choice becomes unimportant and must be made arbitrarily. Extrapolation of any rating curve will lead to greater uncertainty in the predicted concentrations since no data are available to confirm the rating curve shape and no processes are explicitly modelled. Selection of the rating curve most suited to extrapolation then becomes a subjective choice based on the researcher's knowledge of the catchment and expected behaviour. There is of course no guarantee that the appropriate model will be selected in this manner.

4.4.5 MIXING MODELS.

While the above studies tend to indicate that variables other than instantaneous discharge are unimportant they are not studies that have been carried out in catchments which exhibit a high degree of hydrologic variability nor do they explicitly account for different water sources. Hart et al. (1964) in a discussion of

Ledbetter and Gloyna's (1964) study suggest that the solute concentration behaviour of streams in the South-West United States can be explained by separating the stream-flow into surface-flow, Q_s, inter-flow, Q_i, and base-flow, Q_b, components and using a regression equation of the form:

$$C = aQ_s^b + cQ_i^d + eQ_b^f \qquad (4.16)$$

where a, b, c, d, e and f are constants.

A similar approach was taken by O'Connor (1976) who used a mass balance to calculate the concentration of solutes resulting from a mix of surface flow and base-flow. This requires the ratio of surface and base-flow and concentrations for both. O'Connor assumed that concentrations of each were constant and that the ratio of base-flow to total flow, r, is given by:

$$r = 1 \qquad Q < Qo \qquad (4.17a)$$

$$r = Qo^{1-n}\,Q^{n-1} \qquad Q > Qo \qquad (4.17b)$$

where Qo is threshold flow at which surface runoff begins.

O'Connor has applied this model to a number of rivers in the United States and Canada. The parameters required were either taken from reported values in the literature or estimated from catchment characteristics and a variety of different rivers with catchment areas ranging from 15 km^2 to 60,400 km^2 and mean annual runoff ranging from 9.5 mm/a to 1900 mm/a. The models were applied to individual species as well as total dissolved solids. O'Connor also developed a number of more complex models which allowed for spatial and temporal variation.

The impact of various flow sources has also been considered by Pionke et al. (1972) in a regression analysis of five streams in South-Western United States. The relationship used included the ratio of surface-flow to base-flow and dilution of base-flow as the base-flow increased and the regression equation was of the form:

$$\ln C = K' - a \ln Q_b - b\, Q_s/Q_b \qquad (4.18)$$

where a, b and K' are constants. Pionke et al. found that this relationship was superior to the Ledbetter and Gloyna (1964) relationship for explaining the solute behaviour of ephemeral streams in the South-Western United States; however for more reliable streams there was little difference between the two relationships.

Another similar approach to predicting water quality has been taken by Haith and Tubbs (1981) and extended by Haith and Shoemaker (1987). This involves predicting runoff from various sources using the United States Soil Conservation Service Curve Number Equation and assigning a concentration to that flow on the basis of land use and soil type. Base-flow is also predicted and a concentration is assigned to it. By conservatively mixing water from each source an estimate of solute concentration based on spatially distributed catchment properties can be made.

By treating different sources of water separately the above models attempt to include those processes which influence the water balance and flow of water from a catchment surface to its outlet. Generally they do so at the whole catchment-scale. Therefore such models neglect spatial and temporal variation in the concentration of water from various sources. Like rating curves, these models are essentially empirical and as such the comments in §4.4.4 apply. They also require the separation of the hydrograph into various flow components.

4.4.5.1 Base-Flow Separation.

A number of models which require separation of the flow hydrograph have been reviewed above. Pionke et al. (1972) used a digital separation technique. On a rising hydrograph the base-flow was increased by a fixed percentage of flow provided the change in flow was greater than that percentage of flow. On a falling hydrograph base-flow was decreased by 5% of the change in flow. O'Connor (1976) assumed a fixed relationship between flow and base-flow and Hart et al. (1964) did not provide details of their separation technique.

Base-flow separation has been reviewed by Nathan and McMahon (1990). There are many techniques available for base-flow separation which are based on qualitative physical reasoning but for which the quantitative separation involves some arbitrary assumptions. There are two general groups of separation techniques. The first assumes that the base-flow responds at the same time as the surface runoff and the second allows for bank storage by continuing the base-flow recession for some period after the surface runoff begins. Nathan and McMahon examined two separation techniques which could be easily computerised. The first was a smoothed minima technique developed by the Institute of Hydrology (1980) and the second utilised Lyne and Holick's (1979) recursive digital filter .

Compared to the smoothed minima technique, the digital filter technique resulted in larger base-flows in small flashy catchments. This was felt to be a more realistic separation than that provided by the smoothed minima technique. The digital filter also provided a more stable estimate of the base-flow index which was defined as the ratio of base-flow to total stream-flow for the study period. The two automated techniques were also compared to manual separation techniques for five catchments. This comparison indicated that the digital filter provided results which were in closer agreement with the manual techniques. While these techniques and many manual techniques based on analysis of stream hydrographs are essentially arbitrary, they do provide separations that are, at least qualitatively, physically realistic (Nathan and McMahon, 1990) and could be used to provide approximate base-flow separations for use in modelling solute concentrations. Of the two techniques Nathan and McMahon conclude that the digital filter is the better method.

Chapman (1991) in a comment on the Nathan and McMahon paper points out that the Lyne and Hollick recursive digital filter implies that base-flow remains constant after direct runoff ceases. This can be seen from the following manipulation of the filter equations. The equation used in the Lyne and Hollick filter is:

$$Q_{s\,t} = \alpha\, Q_{s\,t-1} + \frac{(1+\alpha)}{2}\,(Q_t - Q_{t-1}) \qquad (4.19a)$$

subject to

$$Q_{s\,t} \geq 0 \qquad (4.19b)$$

where Q_s is the quick response (surface runoff and inter-flow), Q is the total flow, α is the filter parameter and the subscript t refers to time. Equation 4.19a is applied subject to non-negative quick response (Equation 4.19b) and base-flow, Q_b, is calculated from:

$$Q_{b\,t} = Q_t - Q_{s\,t} \qquad (4.20)$$

If the total flow is eliminated from Equations 4.19 and 4.20 the base-flow is given by:

$$Q_{b\,t} = Q_{b\,t-1} + (Q_{s\,t} + Q_{s\,t-1})\frac{(1-\alpha)}{(1+\alpha)} \qquad (4.21).$$

which implies constant base-flow after runoff ceases. Of course, the constraint (Equation 4.19b) ensures that base-flow reduces with time and therefore ensures the filter performs as expected during times of zero surface runoff.

Chapman proposes the following modified filter:

$$Q_{s\,t} = \frac{(3\alpha - 1)}{(3-\alpha)}\,Q_{s\,t-1} + \frac{2}{(3-\alpha)}\,(Q_t - \alpha Q_{t-1}) \qquad (4.22)$$

again subject to Equation 4.19b. This filter includes base-flow recession and the filter constant now has hydrologic significance as the base-flow recession constant. This suggests that the filter constant can be selected objectively by equating it with a base-flow recession constant derived for the catchment (Chapman, 1991). Since this modification is apparently more hydrologically realistic (Nathan and McMahon, 1991) it will be used for any base-flow separation in this study.

4.4.6 STORAGE MODELS.

Up to this point we have not explicitly considered the variation of solute stored in the catchment and the impact that this can have on the solute concentration behaviour of the catchment. Johnson et al. (1969) (§ 4.2.1.) used the concept of a mixing volume in which the amount of solute stored was linked to the storage of water in the catchment. It is also possible to incorporate a solute store in a model which is augmented from a source and depleted by flow from the catchment but otherwise operates independently of the water storage in the catchment.

Clarke and Crawley (1987) developed a lumped catchment model for prediction of concentrations of Oxidised Nitrogen, Total Kjeldahl Nitrogen, Soluble and Total Phosphorus and Turbidity which was based on rating curves modified by catchment solute storage. The model, called Nutmod, predicts concentration by assuming that concentration is proportional to solute storage. This is similar to the method used by more complex simulation models such as SWMM (Huber and Dickinson, 1988). Concentration is predicted by:

$$C = S.\ fn(Q) \tag{4.23}$$

where S is the solute storage for the time period and fn(Q) is the rating curve equation. The solute storage is operated via a simple mass balance system.

Nutmod has been applied to various catchments in the Adelaide Hills, South Australia. The solute supply was set at a constant rate and, using a daily time-step and mean daily flows, daily loads were calculated. The model was optimised by minimising the sum of the absolute errors in calculated and observed cumulative load. The existing quality data, which were discontinuous, had to be converted to daily loads to perform the calibrations. No verification of the model was performed.

4.4.7 CHEMICAL SIMULATION MODELS.

Gutteridge Haskins and Davey (1991) prepared a review of water quality models for the New South Wales Department of Water Resources which include a number of chemical simulation models. Chemical simulation models such as HSPF (Johanson et al., 1984), SWMM (Huber and Dickinson, 1988) or CREAMS (Knisel, 1980) simulate water quality by modelling pollutant availability, wash-off, transport and attenuation. Solute availability is modelled using models of the chemical behaviour of solutes or empirical relationships. While these models are purported to be capable of predicting water quality behaviour, they are highly complex and data intensive and are therefore not considered suitable for this project.

4.5 WIMMERA CATCHMENT.

Flows in the Wimmera river at Glynwylln are characterised by a mean annual discharge of 67 700 Ml/a (RWC, 1990) which equates to 49.9 mm mean annual runoff. The coefficient of variation is 0.88. The flow-weighted mean salinity is 422 EC units and the median salinity is 2 400 EC units (Hooke, 1991). These figures indicate that the flow regime is highly variable and that the catchment is characterised by relatively saline flows. The difference between the flow-weighted mean salinity and median salinity is a function of both the variability in flow and variability in salinity.

In catchments in the South-Western United States (Ledbetter and Gloyna, 1964; Hart et al., 1964; Pionke et al., 1972) which also exhibit variable flows and high salinities, it has been found the solute concentration behaviour of streams has not been well explained by considering discharge alone. This indicates that factors other than discharge may also be important in the Wimmera River.

The above literature review canvases several possible methods for predicting the quality of inflows to the Wimmera River. These include a rating curve, a rating curve adjusted to interpolate between sample points (Walling's Interpolation Model, Equation 4.11) and regression models incorporating antecedent flows or base-flow. A model explicitly incorporating solute storage, and models purporting to simulate the physical and chemical processes which determine water quality were briefly discussed. While a more sophisticated model might be expected to produce more accurate results it also has the disadvantage of being more complex and data intensive. For the purposes of this study a straight forward procedure capable of predicting realistic salinities for surface water inflows to the Wimmera River is required. Therefore methods utilising regression relationships are the most appropriate.

4.5.1 COMPARISON OF METHODS.

4.5.1.1 Preliminary Comments.

All regressions in the following analysis were performed using StatView (Feldman et al., 1987) which is a statistical package for the Apple Macintosh incorporating standard Ordinary Least Squares Regression techniques and Stepwise Regression techniques.

Some of the models compared below rely on multiple regression and a brief discussion of two aspects of this is required. Multiple regression assumes that the explanatory variables are independent or that there is no multicollinearity (Yamane, 1973). Multicollinearity arises when there is high correlation between one explanatory variable and a linear combination of the other explanatory variables. Problems with estimating regression coefficients can arise when there is significant multicollinearity. These problems are manifest in imprecise or highly variable regression coefficients (Yamane, 1973). The variability of regression coefficients is higher than it would be without multicollinearity so t-tests underestimate the significance of individual explanatory variables. On the other hand the residuals about the regression line tend not to be affected, so F-tests, which are based on the residuals, tend not to be affected (Harrison and Tamaschke, 1984). Therefore F-tests can be used with reasonable confidence.

Multicollinearity can be tested for by performing a series of linear regressions between the explanatory variables. A high coefficient of determination then indicates multicollinearity. High correlation between any two explanatory variables is also indicative of multicollinearity. Harrison and Tamaschke (1984) note that the emphasis when considering multicollinearity is on high correlation and correlations of 0.6 to 0.7 should not create undue problems. Where a pair of explanatory variables has a correlation, r, greater than 0.7 one of the pair will be eliminated from the analysis and significance tests will be performed using F statistics.

Another technique used in the selection of variables for a multiple regression model is stepwise regression. Stepwise regression is described by Walpole and Myers (1978) and is a forward selection technique based on partial F ratios. Partial F ratios test the significance of individual variables in a regression. Critical values of the ratio are obtainable from F distribution tables. The following procedure is used in stepwise regression:

(1) the variable that is most significant (has the highest partial F ratio) is selected as the first variable for the regression provided it is statistically significant;

(2) of the variables not included in the model the next most significant is then selected provided it is statistically significant;

(3) a check is made of each variable included in the model to ensure that it is still statistically significant;

(4) steps 2 and 3 are repeated until there are no more statistically significant variables remaining in the group not included in the model.

4.5.1.2 Model Comparison.

A regression analysis of the monthly conductivity data for the Wimmera River at Glynwylln (415206) indicates that 60% of the variation in conductivity can be explained by discharge when a log-linear model is used. A summary of this analysis is included in Table 4.1 and Figure 4.3 shows the relationship between discharge and salinity. A Durbin-Watson test (Yamane, 1973) shows that there is significant serial correlation between the residuals at the 5% level. This suggests that there are processes other than simple dilution which are important in determining the behaviour of solute concentrations, that these processes are linked to the catchment history and that they operate on a time-scale that is significantly longer than the monthly sampling interval. Examination of Figure 4.3 also indicates that there is a tendency for the log-linear model to underestimate conductivity when discharge is low or conductivity high. However use of a second order polynomial did not significantly improve the salinity predictions.

Since there are apparently long-term processes involved in behaviour of water quality in the Wimmera River at Glynwylln, models capable of simulating these effects may be expected to produce more accurate results. For this reason several regression models were assessed including models which incorporated antecedent flows and base-flows. For this assessment the monthly Victorian Water Quality Monitoring Network data for the Glynwylln gauging station were divided in two by taking every second data point. This provides two independent, random data sets with a 2 month sampling interval spanning the period September, 1975 to June, 1988. The ability of various methods to predict one set from the other were then compared.

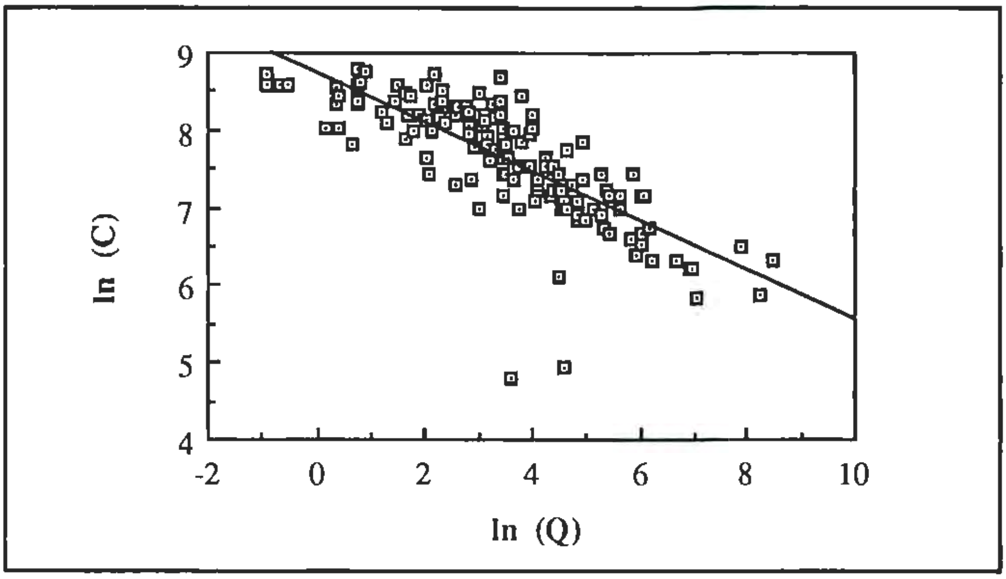


Figure 4.3: Relationship between discharge (Q) and salinity (C) for the Wimmera River at Glynwylln.

Model	EC = 6323 * $Q^{-0.318}$
n	148
R^2	0.602 (ln domain)
Standard Error	0.478 (ln domain) -38%, +61% (real domain)
Durbin Watson Statistic	1.577

Table 4.1: Log-Linear rating model for Wimmera River at Glynwylln calculated using Victorian Water Quality Monitoring Network Data (monthly sample interval).

The basic functional form of a log-linear rating curve was chosen for two reasons. It is a simple form readily amenable to regression analysis in the log domain and it is also a reasonable, though not perfect, model for the relationship between flow and salinity at Glynwylln as can be seen from Figure 4.3. Since the purpose of the models is to predict conductivities in the real domain the comparisons should be made in the real domain (McCuen et al., 1990). Apart from the log-linear model four other models were considered. These included a log-linear model in which predictions are interpolated from neighbouring sample points (Walling, 1984), the Minimum Variance Unbiased Estimator (MVUE) proposed by Cohn et al. (1989), a model incorporating base-flow and a model incorporating total flow and the ratio of base-flow to total flow. A model incorporating antecedent flows was also considered but was eliminated due to high correlation between the explanatory variables.

			Cum. Error		C_{eff}	
Method	Model	R^{2a}	Salinity	Load	Salinity	Load
Log-Linear	$EC = 6209\ Q^{-0.312}$	0.70	7.4%	4.6%	0.61	0.92
MVUE	$EC = 6209\ Q^{-0.312}$ with bias adjustment	N.A.	-1.8%	-3.5%	0.59	0.93
Interpolated Log-Linear	$EC = K\ Q^{-0.312}$ K interpolated	N.A.	-4.4%	-4.1%	-0.21	0.87
Base-flow	$EC = 6300\ Q_B^{-0.332}$	0.62	6.2%	-23.9%	0.64	0.96
Base-flow index	$EC = 13100\ Q^{-0.399} * e^{-0.724B}$	0.63	4.7%	-14.4%	0.64	0.89

Table 4.2: Comparison of errors associated with different solute rating models for the Wimmera River at Glynwylln. [a] Regression Coefficient of determination (in log domain).

The models incorporating base-flow characteristics were developed using multiple regression as follows. Both the total flow, Q, and base-flow, Q_b, were considered as variables to represent the dilution effect. The logarithms of these flows are highly correlated (r = 0.96), therefore only one of the two could be incorporated in a particular model. Since the dilution effect is important and could conceivably be represented by either ln Q or ln Q_b, it was decided to fit two models, one incorporating either variable.

The instantaneous base-flow index, $B_b=Q_b/Q$, was chosen to represent the relative importance of base-flow. Two models (Table 4.2) were then developed from two groups of variables: ln Q, B and ln B; and ln Q_b, B and ln B. Stepwise regression was used and the condition that only one of Q_b/Q and ln Q_b/Q could be incorporated in the model was imposed because these two variables are related.

Conductivities were predicted for the second data set using the five models. The predictions are compared to the measured conductivities in Table 4.2 and Figure 4.4. Table 4.2 provides percentage cumulative errors for the predictions of salinity and load. The coefficient of efficiency (Equation 4.24) (Aitken, 1973) is provided also. The coefficient of efficiency provides an estimate of the variation about the 45° line.

$$C_{eff} = (1 - \frac{SSR}{SSM}) * 100 \qquad (4.24)$$

In 4.24 C_{eff} is the coefficient of efficiency, SSR is the sum of squared prediction residuals and SSM is the sum of squared residuals from the mean.

Larger errors are associated with larger conductivities due to the re-transformation from the log domain so the cumulative error in the predicted conductivity tends to be an indication of the goodness of fit of the models at higher conductivities. On the other hand, the cumulative error in load will reflect errors in the lower conductivity or high flow range because the high flow magnifies errors in conductivity predictions. C_{eff} indicates the variability of the conductivity and load predictions. Again there will be a tendency for the variability in conductivity predictions to reflect the variability in the higher conductivity region and the variability in load to reflect those in the lower conductivity or high flow region.

Several observations can be made from the graphs in Figure 4.4. Firstly there is a tendency for all the models except Walling's interpolation to underestimate the conductivity between 2 000 and 4 000 EC units but for the log linear model to be a good representation outside this range. This corresponds to a flow range of approximately 4 - 40 Ml/d. Secondly the models incorporating base-flow characteristics tend to overestimate slightly conductivities for flows exceeding 40 Ml/d.

Examination of Table 4.2 reveals several points about the different models. Firstly, the inclusion of additional variables in the models incorporating base-flow characteristics does not necessarily lead to an improvement in prediction of conductivity even though the additional variables are statistically significant. If the prediction of load is the ultimate aim then the quality of prediction of concentration, especially the percentage of explained variance, is not necessarily a good indication of a model's ability to predict load. In fact, in this comparison, the three models which explained the most variance in concentration also had the largest errors in cumulative load. Foster (1980) also comes to this conclusion. The reason for this result is clear if the tendency to over-predict conductivities at high flows is noted. For this reason the models incorporating base-flow characteristics will not be used for infilling conductivity data.

Secondly, the negative C_{eff} for Walling's interpolation model indicates that the interpolation is actually increasing the variability of the prediction compared to using the mean EC as the predicted EC. At the same time the cumulative error is reduced. One plausible interpretation of this result is that the short term variation in water quality behaviour is increasing the variability of the predictions. This may occur because short term variability is built into the prediction by the use of interpolation between individual data points. Simultaneously, the long term variation is being included in the predictions and reducing the cumulative error.

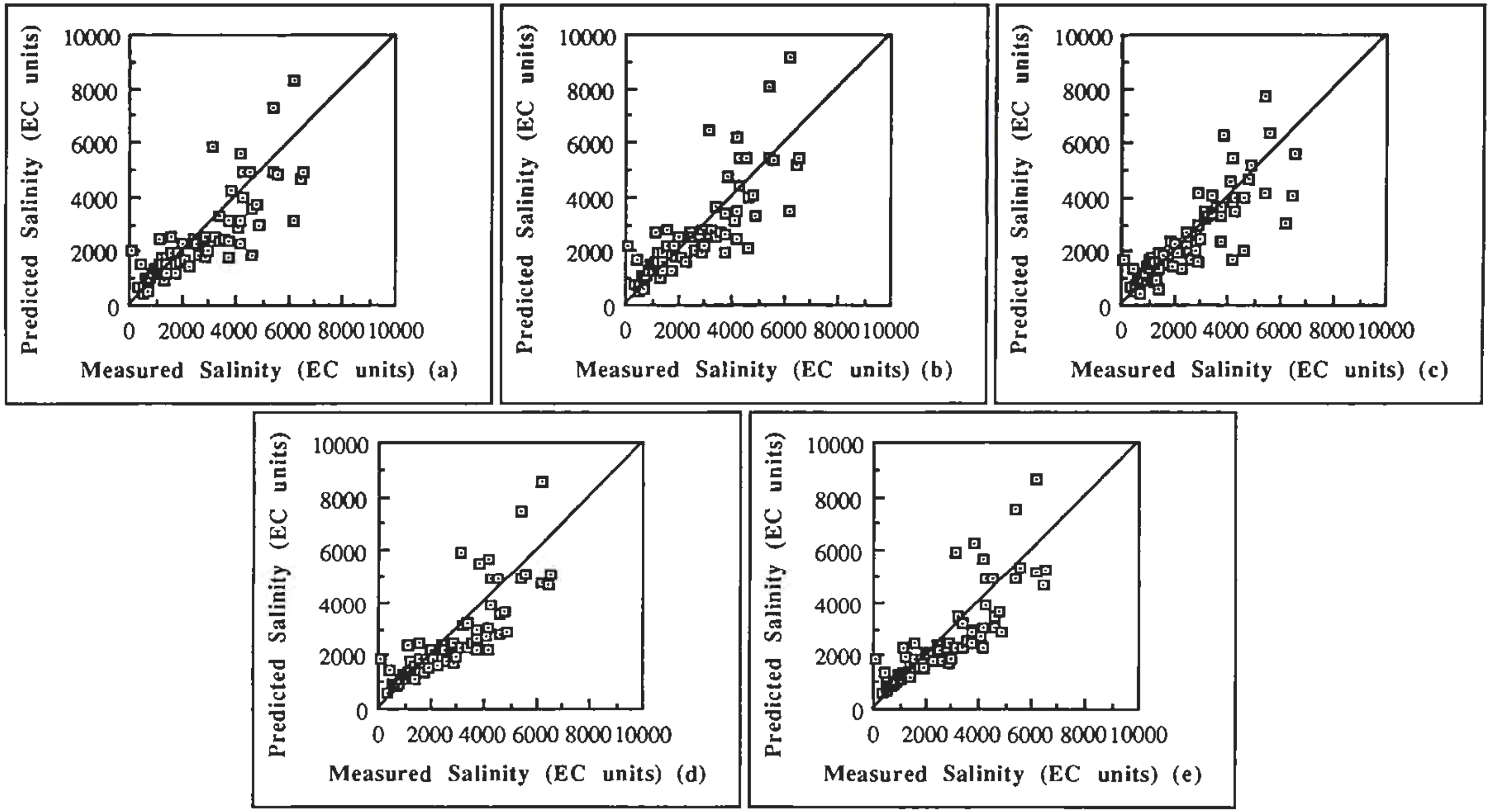


Figure 4.4: Comparison of predicted and measured salinity for different solute rating curve models for the Wimmera River at Glynwylln: (a) Log-linear model; (b)Minimum Variance Unbiased Estimator; (c) Walling's interpolation model; (d) Base-flow model; and (e) Total flow / base-flow index model.

This would be possible if the short term variability was random and of a similar order of magnitude as the long term variation.

One point that should be made is that in practice Walling's interpolation model would be used to interpolate conductivities for the whole period between monthly samples; however in the above comparison Walling's model was used to calculated a conductivity at the midpoint between bimonthly samples. Therefore the application of the model would be under conditions different to those under which it was tested. This problem would not affect the other models as they do not rely on neighbouring data points to predict conductivity.

Finally, of the three models based on the log-linear rating curve, the MVUE provides the most accurate and precise predictions of conductivity and load. This is due to the removal of the bias arising from re-transformation from the log domain which was discussed in §4.4.2.2. The variability of the predictions of the log-linear rating curve and MVUE are similar and significantly less than Walling's interpolation model. Of the six models considered the MVUE is therefore the most appropriate for infilling conductivity data at Glynwylln.

4.6 CONDUCTIVITY IN UNGAUGED STREAMS.

Section 4.3 reviews the more important processes determining the behaviour of stream solute concentration. It is clear that stream solute concentrations are highly variable. Part of this variability is associated with variation in total flow. This is related to hydrologic variability. The other part is associated with variation in processes which affect the contribution of different flow sources, which is also related to hydrologic variability, and with variation in the availability of solute. In other words, while it may be expected that catchments with similar climatic, physiographic and geologic conditions will exhibit similar discharge and solute concentration behaviour; it would also be expected that there would be a greater difference in solute behaviour than in discharge behaviour. The simple reason for this is that catchment discharge integrates the flow determining processes while solute concentration integrates both the flow determining processes and the processes determining the solute concentration of each flow source. This additional variability makes estimation of solute concentration more difficult than the estimation of flow for ungauged catchments.

Log-linear solute rating curves were estimated for ungauged streams by estimating the slope (the power b in Equation 4.3) and the coefficient (a in Equation 4.3) separately. The slope of the rating curve is largely determined by the systematic

Station	n	R^2	b	Confidence Interval	
				Lower	Upper
415206	274	0.76	-0.31	-0.34	-0.28
415207	261	0.75	-0.30	-0.33	-0.26
415237	75	0.67	-0.31	-0.39	-0.23
415238	42	0.54	-0.24	-0.36	-0.12
415244	31	0.59	-0.12	-0.18	-0.06
415245	40	0.84	-0.34	-0.41	-0.26
415252	18	0.89	-0.34	-0.44	-0.25
415253	12	0.23	-0.23	-0.23	0.11

Table 4.3: Solute rating curve slope, b, for eight unregulated streams in the Wimmera River Catchment. The number of salinity samples, n, used in the regression, the coefficient of determination and the 95% confidence limits for b are also provided.

dilution of base-flow by surface flow as total flow increases. This is linked to the systematic part of the hydrologic response to storms. If two catchments are hydrologically similar then, from the above arguments, it may be expected that the slope of the rating curves for the two catchments would also be similar.

If it can be shown that hydrologically similar catchments do indeed exhibit similar rating curve slopes then a rating curve can be fitted to an ungauged catchment simply by transferring the rating curve slope from a similar catchment and then predicting the coefficient, a, in Equation 4.3. If the total load from a catchment without salinity data can be estimated and the flow hydrograph and rating curve slope are known then the coefficient, a, can be calculated so that the estimated and predicted loads are the same.

Log-linear solute rating curves were fitted to data from eight unregulated streams in the Wimmera River Catchment (Table 4.3). The 95% confidence intervals for b contain -0.3 for six of these eight catchments. Of the remaining two catchments, the log-linear relationship is not statistically significant (5% significance level) for Pleasant Creek (415253) and b is significantly greater than -0.3 for Shepards Creek (415244). Shepards Creek drains a small forested catchment on the upper slopes of Mt Cole and may therefore be expected to behave differently from the other streams which drain cleared catchments. It was therefore assumed that a rating curve slope of -0.3 is applicable to all ungauged tributaries.

Hooke (1991) provides the annual salt load per unit area for various unregulated streams in the Wimmera Catchment. The median salt load per unit area is

14.5 t/a.km^2 and the range is from 6 t/a.km^2 to 22 t/a.km^2. In the absence of better information a salt load of 14.5 t/a.km^2 was used to calculate salt loads for ungauged streams in the Wimmera River Catchment. Two exceptions are Norton and Daragan Creeks. These streams drain from the Grampians into the Wimmera River downstream of Horsham. Two salinity samples collected from Nortons Creek indicate that it is significantly less saline than streams in the upper Wimmera Catchment. Therefore a rating curve with slope of -0.3 and coefficient set to reproduce the sample data was used. For Daragan Creek the coefficient of the solute rating curve was set such that the salt load per unit area was the same from both Norton and Daragan Creeks.

This approach is far from ideal. No study of this method has been conducted and, apart from the intuitive argument presented above, there is little evidence to support its use. However it is relatively simple, it provides estimates of the salinity of flows entering the Wimmera River that vary with flow in a manner which is qualitatively correct, and it will also provide plausible salt loads from ungauged catchments.

4.7 ESTIMATION OF DATA FOR SPECIFIC SURFACE WATER INFLOWS TO THE WIMMERA RIVER.

4.7.1 GLYNWYLLN.

Discharge is continuously monitored at Glynwylln. Instantaneous flows were extracted from this continuous record every 6 hours and used to specify the inflow to the MIKE 11 model at Glynwylln. Continuously monitored salinity data were only available for the period 20/6/92 to 31/12/93. Therefore salinity had to be estimated if the MIKE 11 model was to be applied (in this or subsequent studies) to any period prior to 20/6/93. A MVUE model (Equation 4.13), which was based on 13 years of spot salinty data, was used to estimate the salinity from the measured discharge. The parameters used in the MVUE model were: a = 8.734, b = -0.312, s = 0.454, $\overline{\ln Q} = 3.46$, $\sum(\ln Q_i - \overline{\ln Q})^2 = 37.0$ and n = 73.

4.7.2 UNGAUGED TRIBUTARIES.

4.7.2.1 Mt William Creek and The Main Central Channel.

Figure 4.5 shows a schematic representation of the WMSDS and Wimmera River near Lake Lonsdale and Glenorchy. Flows from Lake Lonsdale, Q_{203}, to the Mt William Creek are monitored continuously and the salinity of this flow is measured monthly. Lake Lonsdale is sufficiently large to smooth fluctuations in the salinity

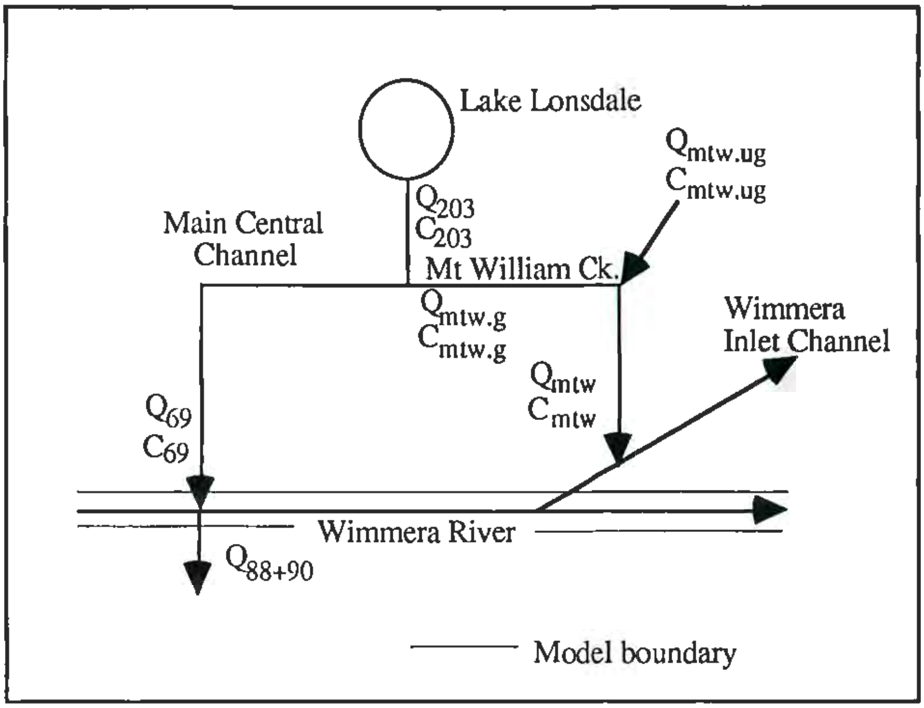


Figure 4.5: Estimation of flow and salinity for the Main Central Channel at Glenorchy and the Mt William Creek at the Wimmera Inlet Channel.

of inflows. Therefore daily, C_{203}, salinity at Lake Lonsdale was estimated by linearly interpolating between the monthly salinity measurements. Water from Lake Lonsdale flows into the Wimmera River at Glenorchy via the Main Central Channel and into the Wimmera Inlet Channel via the Mt William Creek.

The flow into the Wimmera River at Glenorchy is monitored daily, Q_{69}, and these data were used to specify the inflow to the MIKE 11 model at Glenorchy. The salinity of this inflow was assumed to be the same as C_{203}. Monitored outflows from the Wimmera River to the Main Central Channel, Q_{88+90}, at Glenorchy were available and used. Salinity was not required for this flow since it is an outflow from the MIKE 11 model.

Flow from Lake Lonsdale to the Wimmera Inlet Channel was estimated as $Q_{mtw.g} = Q_{203} - Q_{69}$ subject to $Q_{mtw.g} \geq 0$. Flows from ungauged tributaries entering the Mt William Creek and the Wimmera Inlet Channel were estimated as $Q_{mtw.ug} = 0.8*Q_{237}$, where Q_{237} is the discharge in Concongella Creek, and were added to $Q_{mtw.g}$. The salinity of flows from the ungauged tributaries was estimated as $3475*Q_{mtw.ug}^{-0.3}$. The salinity of the inflow to the Wimmera Inlet Channel was then estimated as:

$$C_{mtw} = \frac{Q_{mtw.g}*C_{mtw.g}+Q_{mtw.ug}*C_{mtw.ug}}{Q_{mtw}} \qquad (4.25)$$

which assumes water from the two sources is conservatively mixed.

4.7.2.2 Burnt Creek.

Burnt Creek flows into (Q_{223}) and the out of ($Q_{bc.g}$) the Rocklands Channel and then into the Wimmera River (Figure 4.6). The Rocklands Channel carries water from Toolondo Reservoir to Pine and Taylors Lakes. Flows from Burnt Creek are often diverted into the Rocklands Channel and subsequently to Pine and Taylors Lakes. $Q_{bc.g}$ was estimated by making the assumption that as much water as possible is diverted into Pine and Taylors Lakes unless this diversion was closed down, which was assumed when $Q_{55} < 2$ Ml/d, for some reason such as Pine and Taylors Lakes being full. Given that the capacity of the Rocklands Channel is 600 Ml/d (Barlow, 1987) $Q_{bc.g}$ can be estimated using:

$$\text{if } (Q_{55} \geq 2)\ Q_{bc.g} = Q_{223} + Q_8 - 600 \qquad (4.26a)$$

$$\text{if } (Q_{55} < 2)\ Q_{bc.g} = Q_{223} + Q_8 \qquad (4.26b)$$

subject to $Q_{bc.g} \geq 0$. Q_{55} was not used directly in a water balance because this led to $Q_{bc.g}$ being significant during some periods when this was unrealistic, given that the WMSDS is operated to minimise losses to the Wimmera River (R. McIlvena, RWC, Per. Com.). This was apparently due to errors in the measured flows in the WMSDS channels. The salinity of water spilling from the Rocklands Channel was calculated as:

$$C_{bc.g} = \frac{Q_{223}*C_{223} + Q_8*C_{223}}{Q_{223}+Q_8} \qquad (4.27)$$

Ungauged flows into Burnt Creek downstream of the Rocklands Channel were calculated as $Q_{bc.ug} = 0.34*Q_{237}$ and their salinity was calculated as $C_{bc.ug} = 1830*Q_{bc.ug}^{-0.3}$. The flow in and salinity of Burnt Creek at the point where it flows into the Wimmera River were then estimated as:

$$Q_{bc} = Q_{bc.g} + Q_{bc.ug} \qquad (4.28)$$

$$C_{bc} = \frac{Q_{bc.g}*C_{bc.g} + Q_{bc.ug}*C_{bc.ug}}{Q_{bc}} \qquad (4.29)$$

4.7.2.3 Other Tributaries.

Table 4.4 summarises the calculation of flows and salinities for other ungauged tributaries. With the exception of McKenzie Creek flows were determined by scaling the Concongella Creek hydrograph and salinities were determined with rating curves. For McKenzie Creek gauged flows were used where available,

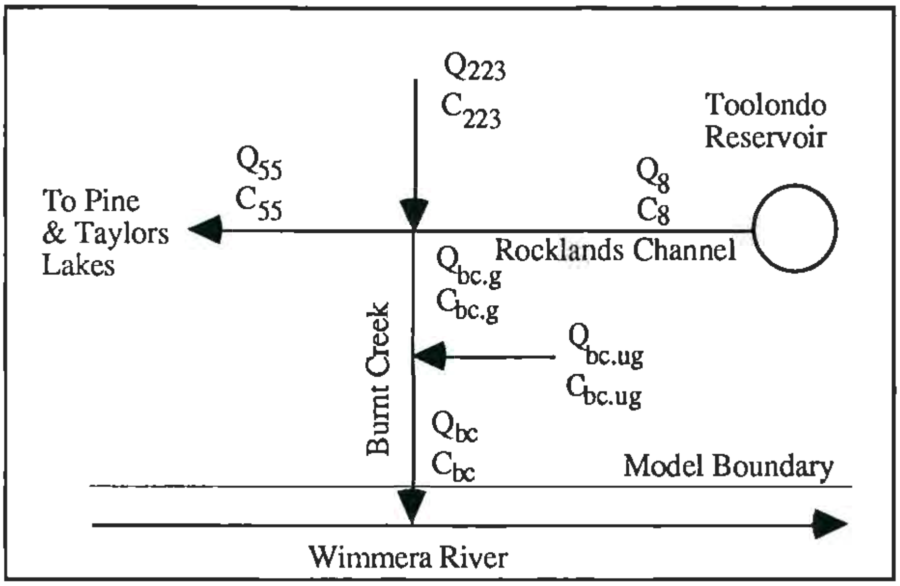


Figure 4.6: Estimation of flow and salinity for Burnt Creek at the Wimmera River.

Tributary	Discharge Estimate	Salinity Estimate
Seven Mile Creek	$Q = 0.49 * Q_{237}$	$414 * Q^{-0.3}$
Concongella Creek	$Q = 1.31 * Q_{237}$	$1034*Q_{237}^{-0.31} + 618*Q_{237}^{-0.3}$
Sheepwash Creek	$Q = 0.54 * Q_{237}$	$80 * Q^{-0.3}$
McKenzie Creek	$Q = 2.413E\text{-}4*Q_{223}^{2} + 0.07*Q_{223} - 0.69$	$328 * Q^{-0.3}$
Norton Creek	$Q = 0.63 * Q_{237}$	$100 * Q^{-0.3}$
Daragan Creek	$Q = 0.30 * Q_{237}$	$80 * Q^{-0.3}$

Table 4.4: Estimated discharge and salinity for Seven Mile Creek, Concongella Creek, Sheepwash Creek, McKenzie Creek, Norton Creek and Daragan Creek at their confluences with the Wimmera River. Q_{237} is the discharge in the Concongella Creek at Stawell and Q_{223} is the flow in Burnt Creek at 415223.

otherwise a regression relationship with the flow in burnt Creek was used. Salinities for McKenzie Creek were calculated with a rating curve.

4.8 GROUNDWATER INFLOWS TO THE UPPER WIMMERA RIVER.

The hydrogeology of the upper Wimmera River is described in this section and estimates of the flow between the Wimmera River and the groundwater between Glynwylln and Glenorchy are made. Issues of errors and representativeness are then discussed. The calculated groundwater flows are compared to other hydrologic fluxes and the contribution of groundwater flows to the salt load in the river between Glynwylln and Glenorchy during low flow periods is calculated. Finally the implications for modelling are discussed.

4.8.1 HYDROGEOLOGY.

Foley (1992) summarises the available hydrogeologic information for the Wimmera River Catchment upstream of Glenorchy. There are two major aquifers present in this part of the catchment. The first consists of coarse gravel and sand deposited above the Palaeozoic basement. This gravel and sand is generally overlain by clay and forms a confined aquifer system. The second is a fractured rock aquifer in the Palaeozoic basement. In the area between Glynwylln and Glenorchy which is the area of interest in this study, the lower alluvial unit is typically an angular quartz gravelly clay interbedded with cemented gravel and course sands and is 20-40 m thick. The clay layer in this area is typically 15 - 25 m thick and consists of interbedded clay, sandy clay and clayey silt. The salinities in the confined aquifer in this area typically increase from 3 000 to 7 000 mg/l TDS as you move away from the river.

Foley (1992) provides potentiometric information based on struck water levels calculated by reducing the depth at which water was intersected during drilling with data from digital elevation models and topographic maps. Monitored bore levels were also used where available. Given the method of reducing water levels, some inaccuracy can be expected. Since the potentiometric information is based primarily on struck water levels; it is indicative of water table levels. Combining these water table levels with information on Wimmera River channel invert levels upstream of Glenorchy indicates that there is potential for groundwater inflow to the Wimmera River for approximately the upper 2/3 of the reach between Glynwylln and Glenorchy. Again given the inaccuracies in water table levels this figure can only be indicative.

Further support for the notion that groundwater may be entering the Wimmera River between Glynwylln and Glenorchy comes from two other studies. A study of stream-flow losses along the Wimmera River (RWC, 1988) indicates that the river gains a significant amount of water between Glynwylln and Glenorchy. Hooke's (1991) analysis of the available salinity data for the Wimmera River catchment indicates that the salt load entering the river from ungauged sources between Glynwylln and Glenorchy is 18 400 t/a. Concongella Creek, which is the only major tributary entering the river in this reach, only contributes 3 400 t/a (Hooke, 1991)(corrected for data inaccuracies). Salt loads from ungauged sources, which include ungauged tributaries and groundwater, are thus quite large which is indicative of a significant groundwater source.

Interaction between the Wimmera River and the groundwater between Glenorchy and Roseneath has not been studied. However it is believed on the basis of groundwater potentiometry, that any flow is from the river to the groundwater over this reach (C. McAuley, RWC, Per. Com.). The stream-flow loss study (RWC, 1988) indicates that there was a small loss between Glenorchy and Faux Bridge, which is just upstream of Roseneath, over the period of the study. This loss actually occurred between Glenorchy and Huddlestons Weir where the entire flow was diverted from the river. The loss was small and could be explained as a result of experimental error. It is therefore likely that there is either no interaction between the Wimmera River and the groundwater between Glenorchy and Roseneath or that the stream is losing water to the groundwater.

4.8.2 DATA AVAILABLE.

All the data used to analyse the interaction between the Wimmera River and groundwater relate to either the flow or salinity of surface waters. Flow data used in this analysis were measured at RWC hydrographic stations and WMSDS monitoring stations. Spot salinity data collected by the RWC hydrographic section during stream gaugings and VWQN salinity data were used. Short electrical conductivity records (20/6/92 to 30/4/93) were also available from data loggers at Glynwylln (415206), Upstream of Glenorchy (415258), Horsham (415200) and from both the channel inflow (69) and outflow (88+90) at Glenorchy when this analysis was conducted.

4.8.3 ESTIMATION OF GROUNDWATER INFLOWS BETWEEN GLYNWYLLN AND GLENORCHY.

The basic methodology used to determine groundwater interaction with the Wimmera River relied upon mass balance with the assumption that inflows of groundwater were equal to the mass imbalance. Periods were chosen when it could be assumed that the only unknown flow into or out of a reach was to or from the groundwater. Both water and salt balances were performed so that an indication of the reliability of the calculated groundwater flow rates could be obtained.

4.8.3.1 Flow Balances.

Continuous discharge data are available at the top and bottom of the reach from the stream gauging stations 415206 and 415201 respectively. Stream gauging data are also available for Concongella Creek (415237) which is the major tributary entering the river in this reach and for Wattle Creek (415238) which is a major tributary which enters the river just upstream of the reach. Channels enter and leave the river at Glenorchy. Discharge data are available for the inflow (69) and for the outflow by combining two monitoring sites (88 and 90) (Figure 4.1). Data are available for all these sites, except 415238, from March 1976 until March 1993. Climatic data are unavailable after 31/12/90. A water year beginning on the 1st of September was chosen so that the start of the water year coincided with the high flow season and the period 1/9/76 to 31/8/90 was used in this analysis.

By examining the data for Concongella Creek and Wattle Creek it was possible to determine periods when neither of these streams were flowing significantly. A flow of less than 1 Ml/d was assumed to be insignificant. These are the two major tributaries to the Wimmera River in the locality of interest. It was assumed that during these periods the flow from other ungauged tributaries would also be insignificant.

Groundwater inflows were then calculated as the difference between surface outflows and inflows. It was clear from the high groundwater flow rates calculated and sudden changes in the calculated groundwater flow rate that, during periods when the channel system was operating, significant errors in the calculated groundwater flows existed. This problem had been anticipated due to the large size of channel flows compared to the river flows during these periods and due to the relatively poor quality of channel flow measurement. The period of analysis was reduced by excluding periods when the channel system was operating.

Flow balances were then calculated using the flow in the Wimmera River at Glynwylln, any small flow in the Concongella Creek ($\leq$ 1 Ml/d) and the flow in the Wimmera River at Glenorchy. An allowance for evaporation from the channel was also made by assuming that evaporation occurred at the potential rate as estimated using the Penman method (Penman, 1948) with the Watts and Hancock (1984) wind function. The water surface area was estimated as the reach length times the average water surface width during low flows which was estimated from data collected by Anderson and Morison (1989e). Figure 4.7 shows the time-series of groundwater flows calculated. The mean groundwater inflow over this period was 2.4 Ml/d.

4.8.3.2 Salt Balances - Continuous Salinity Data.

Continuous salinity and discharge data are available for the Wimmera River at Glynwylln (415206) and at station 415258 which is just upstream of the Glenorchy Weir Pool. Estimates of discharge at 415258 are available from 415201, provided

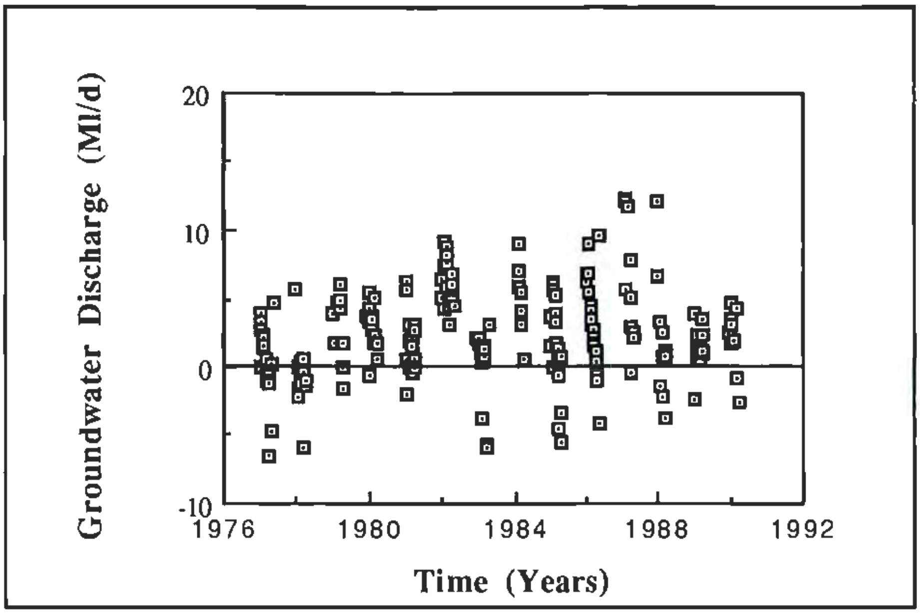


Figure 4.7: Time series of groundwater flows into the Wimmera River between Glynwylln and Glenorchy for low flow periods calculated using flow balances.

the channel system is not operating. At the time of this analysis concurrent salinity and flow data were available for the period from 20/6/92 until 30/4/93 however some data was missing due to data logger failure.

For the low flow periods, the salt load from groundwater inflow was estimated as the difference in salt load at the downstream and upstream ends of the reach. An estimate of the groundwater salinity (3 000 ppm TDS) adjacent to the river was available from Foley (1992). This was used to convert the salt load due to groundwater inflow to a flow rate. Where there was a loss from the reach this loss was assumed to occur at the salinity of the upstream inflow and the calculations were made accordingly. These estimates of groundwater flow rate are shown in Figure 4.8.

4.8.3.3 Salt Balances - Spot Salinity Data.

Another source of data that was used to calculate groundwater interaction was spot salinity data. These data comes from two sources. The first is salinity readings taken concurrently with rating measurements made by the RWC hydrographic section. These data are irregularly spaced in time. The second is the Victorian Water Quality Monitoring Network (VWQN) data which are regular monthly data. Data from both these data sets are available for Glynwylln and Glenorchy.

To estimate groundwater inflows it was necessary to select pairs of salinity measurements from the upstream and downstream stations which were taken at

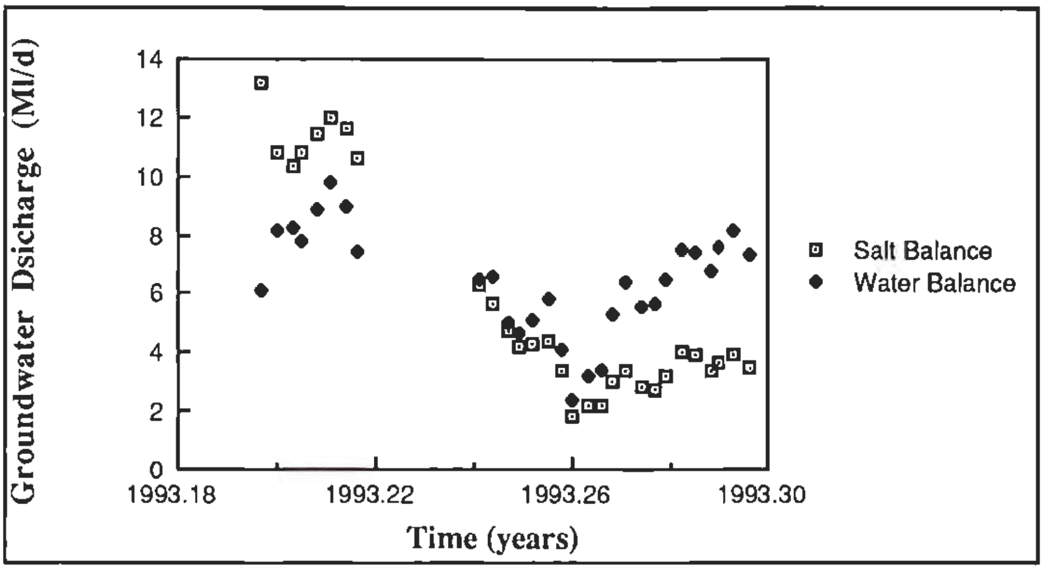


Figure 4.8: Time series of groundwater inflows to the Wimmera River between Glynwylln and Glenorchy during low flow periods calculated using a daily salt balance (continuously recorded salinity) and a daily water balance.

approximately the same time. The selection criterion was that the data points should not be more than a week apart. In some cases both hydrographic and VWQN data existed on the same day. In this case the VWQN data were used since they were based on a laboratory tested sample rather than a field measurement and should therefore be more accurate. It is somewhat disconcerting to note that there is regularly a difference of 10% to 20% between these two data sets. Salt balances were used as in §4.8.3.2 to estimate the groundwater flow rates (Figure 4.9).

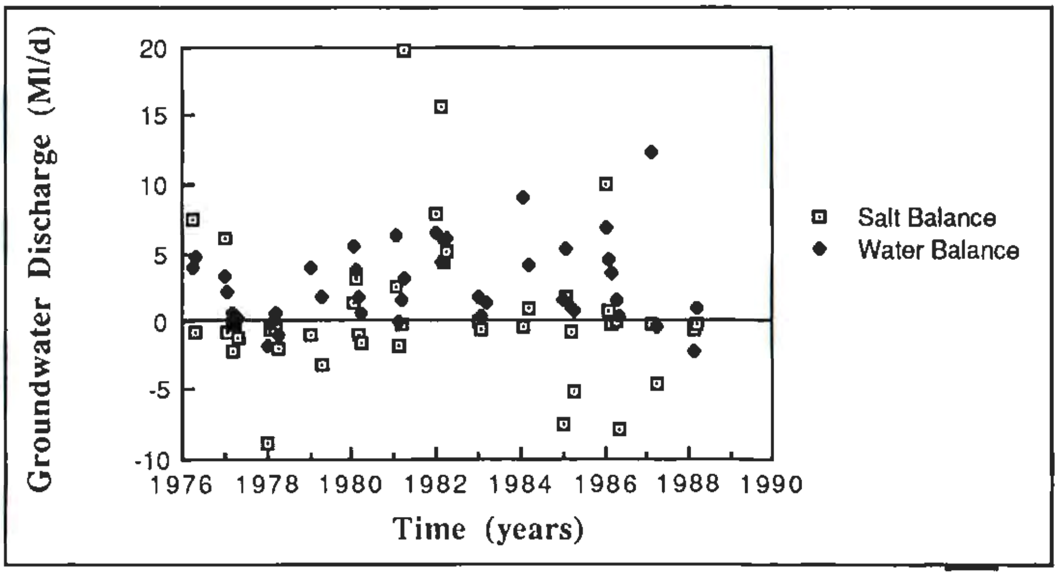


Figure 4.9: Time series of groundwater inflows to the Wimmera River between Glynwylln and Glenorchy during low flow periods calculated using a daily salt balance (spot salinity) and a daily water balance.

4.8.4 COMPARISONS, ERRORS AND REPRESENTATIVENESS.

4.8.4.1 Comparison of Water and Salt Balances.

Groundwater flow rates have been calculated using both water balances and salt balances. The salt balance calculations have been conducted for two different sets of salinity data. These sets of salinity data do not overlap but do provide in each case estimates of groundwater flow rates for periods during which it is also possible to calculate the groundwater flow rate using a water balance. Therefore it is possible to compare the groundwater flow rates calculated by water balance with those calculated using salt balances thereby providing some cross-checking of the calculated flow rates.

Because the calculation of salt load requires the flow rate, the estimates of groundwater flows from the water balance and the salt balance are not completely independent. The proportional contribution of the change in salinity over the reach to each of the estimated groundwater flow rates was calculated using Equation 4.30 and was used to identify periods when the calculated groundwater flow was dominated by a change in salinity ($P_S > 0.5$) rather than flow over the reach.

$$P_S = \frac{(C_2 - C_1)(Q_2 + Q_1)}{(Q_2 - Q_1)(C_2 + C_1) + (C_2 - C_1)(Q_2 + Q_1)} \qquad (4.30)$$

In Equation 4.30 Q refers to discharge and C to salinity, the subscripts 1 and 2 refer to the upstream and downstream ends of the reach respectively and $\mathbf{P_S}$ is the proportion of the calculated groundwater flow attributable to the increase in salinity between 1 and 2.

Figure 4.10 shows a comparison of the groundwater flow rates calculated using the continuous salinity data and those using a water balance. Daily estimates of the groundwater discharge have been used due to the limited amount of continuous salinity data available during low flow periods and evaporation has been ignored in the water balance calculations due to the unavailability of climatic data. The points where the salinity is dominant in the salt balance calculations are differentiated from those where the discharge is dominant. Figure 4.11 shows a similar comparison between the groundwater flow rates calculated from the spot salinity data and those from water balance. Again the points in which salinity is dominant in the salt balance calculations are differentiated from those where it is not.

Table 4.5 compares the mean groundwater discharge rates calculated using salinity balances with the mean groundwater discharge rates calculated using water balances

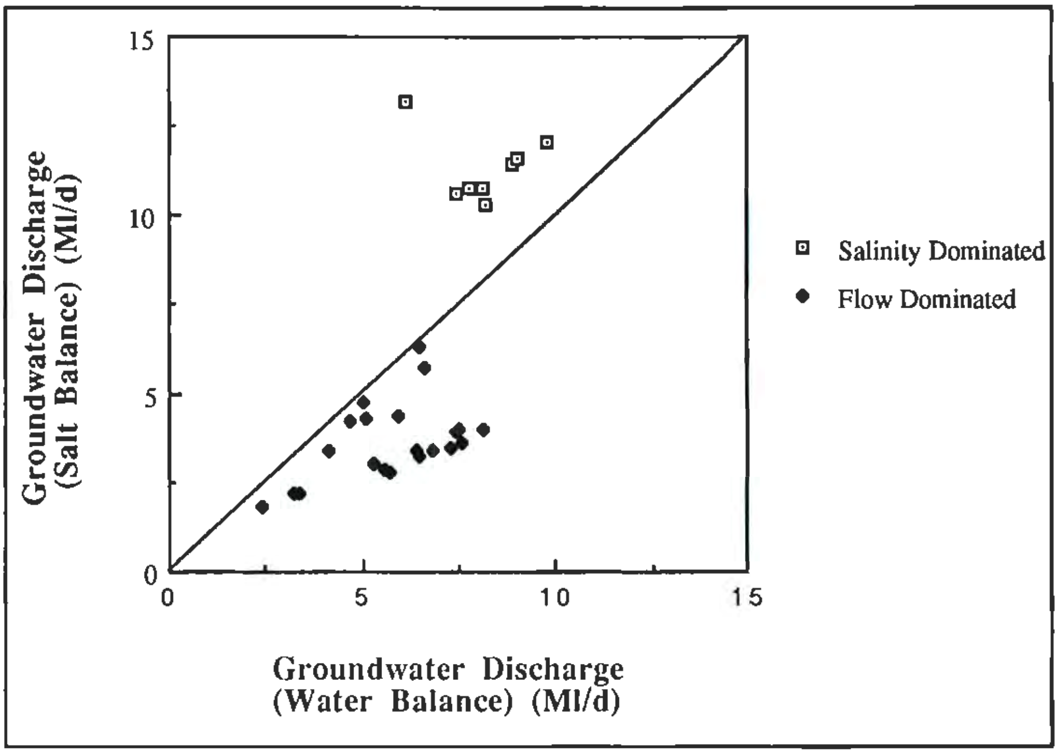


Figure 4.10: Comparison of groundwater inflows to the Wimmera River between Glynwylln and Glenorchy during low flow periods calculated using a daily salt balance (continuous salinity) and a daily water balance. Points where the groundwater flow calculated using a salinity balance is dominated by a change in salinity over the reach are differentiated from those where a change in flow dominates.

for the same periods. The mean flow rate calculated using continuous salinity data is similar to that calculated from a water balance; however the groundwater flow rates based on the salt balances are more variable. A paired t-test indicates that the results from the two methods are not statistically different (5 % significance level). The higher variability resulting from the salt balances is not surprising given that the salt balance calculations include variability in both discharge and salinity while the water balance calculations only include variability associated with the discharge. Figure 4.10 indicates that, where the groundwater flow rates calculated using salt balances are dominated by differences in salinity along the reach, the salt balance calculations tend to result in a higher groundwater flow than the water balance calculations. When the salt balance calculations are dominated by differences in discharge along the reach the opposite tends to be the case.

From Table 4.5 it can be seen that the spot salinity data lead to groundwater discharge rates that are highly variable. Part of the reason for this is related to the longer period and greater variety of conditions for which the spot salinity data is available and part is probably due to the fact that samples were taken several days

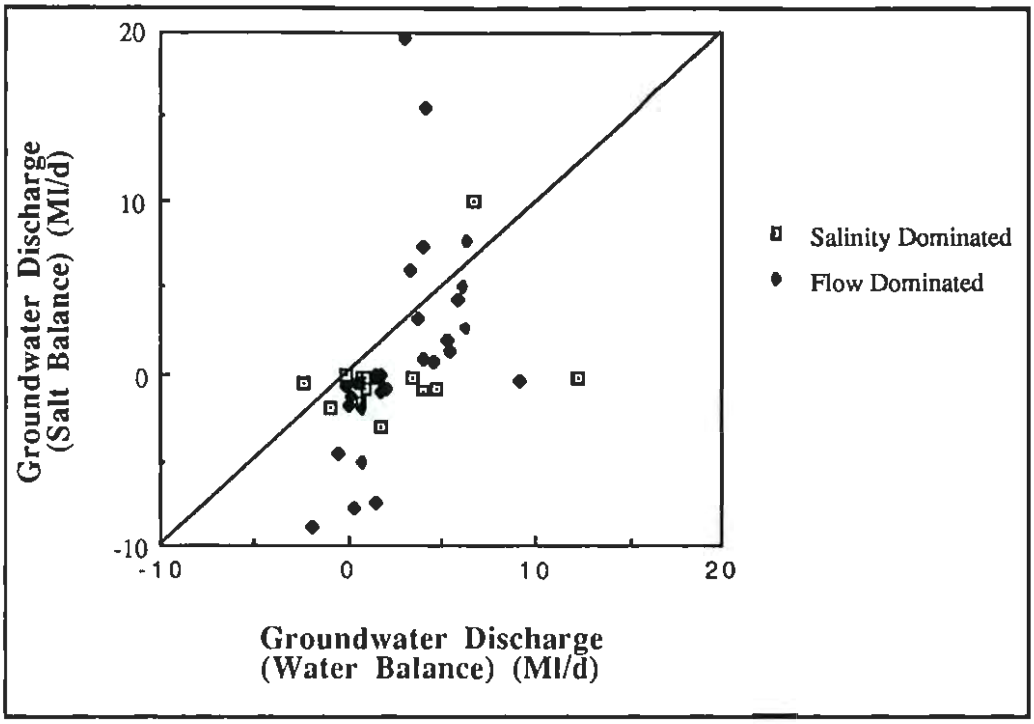


Figure 4.11: Comparison of groundwater inflows to the Wimmera River between Glynwylln and Glenorchy during low flow periods calculated using a daily salt balance (spot salinity) and a daily water balance. Points where the groundwater flow calculated using a salinity balance is dominated by a change in salinity over the reach are differentiated from those where a change in flow dominates.

		Salt Balance		Water Balance	
Comparison	n	μ (Ml/d)	σ (Ml/d)	μ (Ml/d)	σ (Ml/d)
Continuous Salinity	29	5.78	3.65	6.43	1.80
Spot Salinity	44	-0.05	7.66	2.66	2.94

Table 4.5: Comparison of mean and variance of groundwater discharge rates calculated from water balances and salt balances for continuous salinity data and for spot salinity data.

apart and are therefore not strictly comparable. Part may also be due to errors in the salinity data. The groundwater flow rates calculated using the spot salinity are statistically different (paired t-test, 5 % significance level) from those calculated using a water balance.

Evaporation has been included in the water balance calculations for comparison with groundwater flow rates calculated from the spot salinity data. However evaporation has been excluded from the water balance calculations for comparison of rates calculated using the continuous salinity data and water balances. If evaporation was excluded from the water balances which were compared with the spot salinity balances the statistical difference would disappear. On the other hand, if an estimate of the mean evaporation is included in the water balances which were compared with the continuous salinity balances a statistically significant difference would appear. Thus if evaporation is included in the water balance calculations then the results are statistically different from the groundwater flow rates calculated from the salt balances but if evaporation is excluded the statistical difference disappears.

It is tempting to simply ignore evaporation in the calculations; however we know that some loss from the river channel to evaporation must occur so the conceptual model that includes evaporative losses is more correct even if the estimated evaporation rates are incorrect. The mean evaporation rates calculated (1.39 Ml/d) are also significant compared with the groundwater flow rates (1.02 Ml/d excluding evaporation, 2.41 Ml/d including evaporation). Therefore there is a need to examine the possible sources of error that may lead to these results. There are several sources of such error including errors in flow measurement, errors in salinity measurement, an error in the assumed salinity of the groundwater entering the river, errors in the estimation of the evaporative losses and the occurrence of flows from tributaries when these were assumed to be negligible. Systematic errors leading to an overestimate of evaporative losses or an overestimate of the salinity of groundwater entering the river would lead to the groundwater flow rates estimated from the water balance being systematically higher than those estimated from the salt balance. Inflows of relatively fresh surface water that is unaccounted for, would also lead to the water balances indicating a greater groundwater inflow than the salt balances.

It is also possible that storage effects are confounding the salt balance calculations, especially for the spot salinity. For the samples used, the mean of the spot salinity at Glynwylln is 4 150 EC while it is 3 000 EC at Glenorchy. This actually indicates that there is a dilution of the river water by relatively fresh water. Since periods when the major tributaries are flowing have been excluded, it is unlikely that this dilution is resulting from tributary inflows. It could however be a result of storage of relatively fresh water which originated from the WMSDS channel system within the Glenorchy Weir pool. For the continuous salinity data used, the mean salinity at 415206 is 2 760 EC and at 415258 is 2 540 EC so again there is a decrease in salinity downstream. Storage in the Glenorchy Weir pool can't explain the continuous salinity data since that is measured upstream of the Glenorchy weir pool. However if there were significant storage effects in the channel upstream of station 415258 a similar explanation could apply.

Comparison of the groundwater flow rates calculated from salt balance and from water balance are in better agreement when evaporation is ignored. However we would expect some evaporative loss from the channel and therefore it is argued that the inclusion of evaporation is more physically correct. There are a number of sources of error which could account for the apparently inferior results obtained when evaporation is included. Due to these errors it is not possible to eliminate evaporation as a significant part of the water balance and, since estimates of the magnitude of the evaporative component indicate it is important, it is retained.

4.8.4.2 Comparison with 13 Year Balance.

Water and salt balance calculations by Hooke (1991) (corrected for data inaccuracies) for the period from 1976 to 1988 inclusive, indicate that there is an average discharge of 30 Ml/d from all ungauged sources between Glynwylln and Glenorchy and that the flow-weighted mean salinity of this water is 2 800 EC. Typical flow weighted mean salinities at gauging stations around the catchment vary from 50 EC to 580 EC (Hooke, 1991). Therefore this 13 year water and salt balance indicates that there is a highly saline source of water. Thus it is expected that there is a significant amount of groundwater entering the stream over this reach. Given the above figures and assuming the groundwater has a salinity of 5 000 EC and the surface inflows have a flow-weighted mean salinity of 500 EC; a mass balance indicates that the mean groundwater inflow rate is 15 Ml/d. The assumed surface water salinity is at the upper end of the observed range and if a lower value is used the groundwater inflow calculated by this method increases. This is an order of magnitude higher than the groundwater inflow rates calculated in §4.8.3 using flow balances during low flow periods.

4.8.4.3 Comparison with Darcy Flow Estimate.

An estimate of the plausible range of groundwater discharge into the Wimmera River between Glynwylln and Glenorchy can be made using Darcy's equation (Chow, 1964). The formation in which the river channel is situated is interbedded clay, sandy clay, and clayey silt and is 15 - 25 m thick (Foley, 1992). The river channel is approximately 10 m deep. Assuming a hydraulic gradient of between 0.001 and 0.01 towards the river, a zone of capture of 15 m depth (5 m above and 10 m below the river bed) and a 30 km long reach; the groundwater flow into the river in Ml/d would be between $1.K_{sat}$ and $10.K_{sat}$; where K_{sat} is the hydraulic conductivity in m/d. Given the geologic formation, it can be assumed that the value of hydraulic conductivity will be between 10^{-4} and 1 m/d (Chow, 1964). This gives a range of plausible flows of between 10^{-4} and 10 Ml/d. This estimate has assumed that the sand and gravel aquifer below the clay layer is not hydraulically connected to the river at any point in this reach. The estimate of plausible groundwater inflows based on Darcy's equation indicates that all the groundwater flows that have been calculated are realistic from a groundwater flow perspective.

4.8.4.4 Errors.

There are a number of potential sources of error in both the 13 year water and salt balance calculations and in the weekly flow balance described above. For the 13 year water balance the difference between total inflows and total outflows (ie the estimate of ungauged flow) was only 4 % of the total flows used in that calculation, so there is obviously potential for significant errors in the ungauged flow calculated. The same is true of the salt balance; however in this case the ungauged salt load is 20 % of the total monitored salt load. There is of course error in the salt load due to both errors in flow measurement and errors in determining the salinity of the flow. The salt load at a gauging station was determined by calculating the flow-weighted mean salinity from available samples and multiplying by the mean discharge (Hooke, 1991). The errors associated with this calculation are unclear.

Of course the above analysis suffers from a similar problem. The flow imbalance in this case is the groundwater flow, and for the weekly flow balance calculations this is 16 % of the total measured flows plus the estimated evaporation. Since the analysis has been restricted to low flows and the two gauging stations where significant flow occurred have low flow weirs, the errors in measured flow should be smaller than for the annual flow balances which include all high river flows and all channel flows. However, the errors in the evaporative losses used could be significant. Figure 4.12 shows estimated errors for the groundwater flows estimated from weekly flow balances. Errors of 10 % for the discharges and 40 % for the evaporative losses were used in these calculations. Any points on Figure 4.12 which fall within the V have an estimated groundwater discharge which is smaller in magnitude than the error.

It is felt that the groundwater flows calculated in §4.8.3.1 are a more reliable estimate of the interaction between the groundwater and the Wimmera River between Glynwylln and Glenorchy for the following reasons. Firstly, during low flow periods the errors in flow measurement should be smaller and thus the errors in the groundwater flow estimate should be smaller. Secondly, the 13 year balance requires discharges and salinities from three stream monitoring stations and three channel monitoring stations. The channel monitoring stations are thought to be less accurate. Alternatively, the low flow period estimates only require flows from

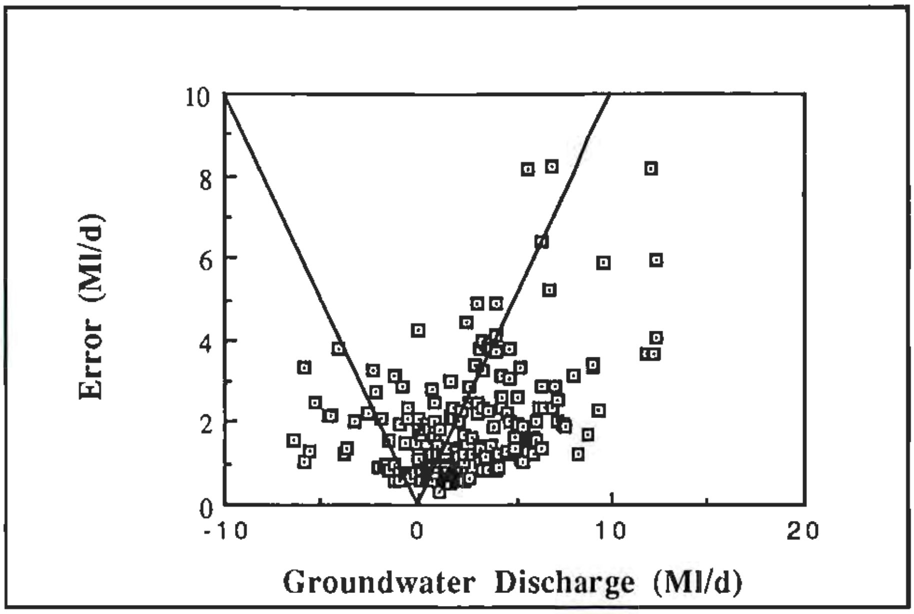


Figure 4.12: Estimated absolute errors in groundwater discharge to the Wimmera River between Glynwylln and Glenorchy calculated using flow balances during low flow periods.

three stream gauging stations which means the error should be smaller. Thirdly, groundwater flows calculated on the basis of the 13 year balances involve determination of both the water and the salt balance and an assumption about the salinity of the ungauged sources of water, whereas the low flow period estimates only rely on the water balance.

4.8.4.5 How Representative are the Periods used?

Figure 4.13 shows the distribution of average weekly flows for the Wimmera River at Glynwylln for the period March 1976 to March 1990. The distribution of flows for the periods used in the analysis is also shown. It can be seen that the flows used in the analysis represent the low flow periods at Glynwylln. Therefore this analysis is only applicable to low flow periods.

4.8.5 REVERSAL OF FLOW BETWEEN THE GROUNDWATER AND RIVER.

Is it reasonable for there to be both inflows and outflows of groundwater along a given river reach during low flow periods as is observed in Figure 4.7? Often the answer is no because the water table is either significantly above or significantly below the river. However in this case the available groundwater information (Foley, 1992), suggests that the level of the water table crosses from above to below the stream invert somewhere between Glynwylln and Glenorchy. Thus there is a potential for groundwater inflow in the upper part of the reach of interest and the possibility of outflow in the lower part of the reach. As water table levels

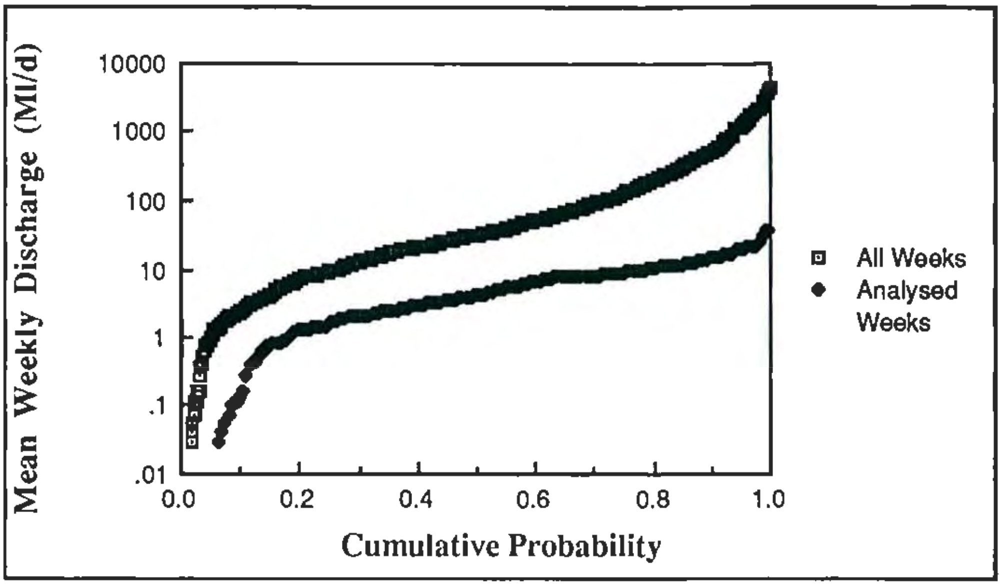


Figure 4.13: Flow duration curves for the entire set of flow data and for the low flow data set.

varied under the influence of seasonal and annual variations in recharge, it may be possible that there would be a change between net recharge from the river to the groundwater and net discharge from the groundwater to the river over time.

4.8.6 RELATIONSHIP TO OTHER HYDROLOGIC FLUXES.

4.8.6.1 Groundwater Flows and River Flow.

It may be expected that there would be some relationship between the river flow and the rate of flow between the river and the groundwater. Two possible factors could lead to such a relationship. Firstly, higher river flows would lead to higher hydraulic heads in the river which would tend to reduce groundwater discharge or increase recharge. The relationship between river discharge and groundwater inflow would have a negative trend. Alternatively, it may be expected that, because higher flow periods would generally be associated with wetter periods, higher water tables and higher base-flows, a positive relationship between river flow and groundwater inflow would exist.

Figure 4.14 shows the groundwater discharge as a function of river flow at the upstream end of the reach. A small positive trend is visually evident; however there is a large amount of scatter. When a linear regression is fitted the coefficient of determination is only 0.02.

4.8.6.2 Groundwater Flows and Rainfall.

A positive relationship between groundwater inflows to the river and rainfall would be expected since higher rainfall periods would be periods of higher recharge and

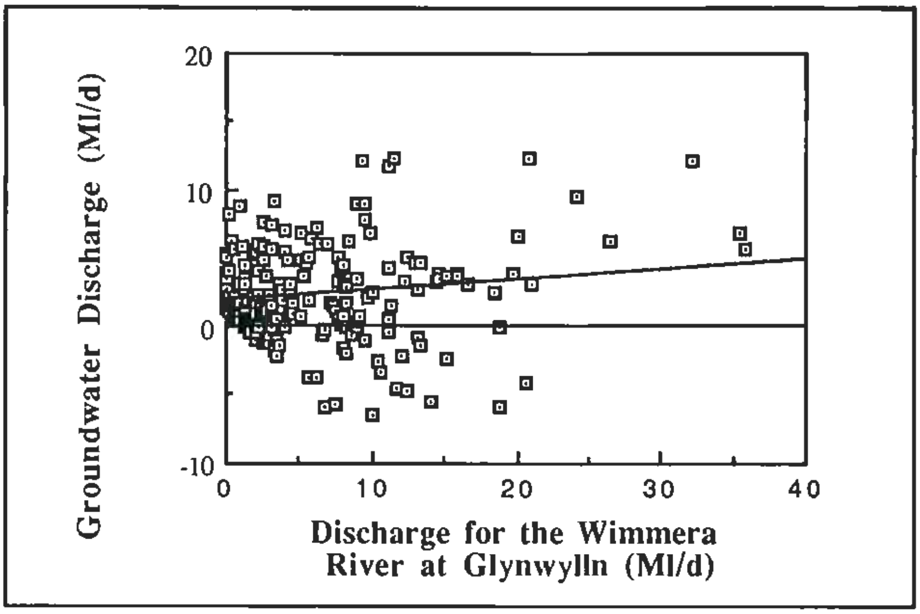


Figure 4.14: The relationship between groundwater discharge to the Wimmera River between Glynwylln and Glenorchy during low flow periods and the discharge in the Wimmera River at Glynwylln. The regression line shown has an R^2 of 0.02.

base-flow. Figure 4.15 shows the groundwater discharge as a function of rainfall accumulated for 30, 90 and 180 days. A positive relationship is evident; however again there is a substantial amount of scatter. The R^2 values are 0.00, 0.05 and 0.19 respectively for the 30, 90 and 180 day accumulated rainfalls.

4.8.7 LOW FLOW SALT LOAD CONTRIBUTION.

The salt load contributed by groundwater inflows was calculated as a percentage of the total inflowing salt load. The salinity at 415206 was calculated using the MVUE model (§4.5) and monitored discharge. The total inflowing salt load was then estimated as the salt load at 415206 plus the salt load associated with any groundwater inflow. If there was actually a flow to the groundwater then this was excluded from the calculation of the total inflow.

Figure 4.16 shows salt load associated with groundwater flows, expressed as a percentage of the incoming salt load, as a function of the river flow at Glynwylln. On average, during the low flow periods used in the analysis, groundwater flows lead to a 30 % increase in the salt load between Glynwylln and Glenorchy; however there is a large amount of variation as is evident from Figure 4.16. This scatter will, at least in part, be due to errors in the estimate of the groundwater inflow rate. Figure 4.16 also indicates that there is a reduction in the importance of the groundwater contribution to the salt load as the river flow increases.

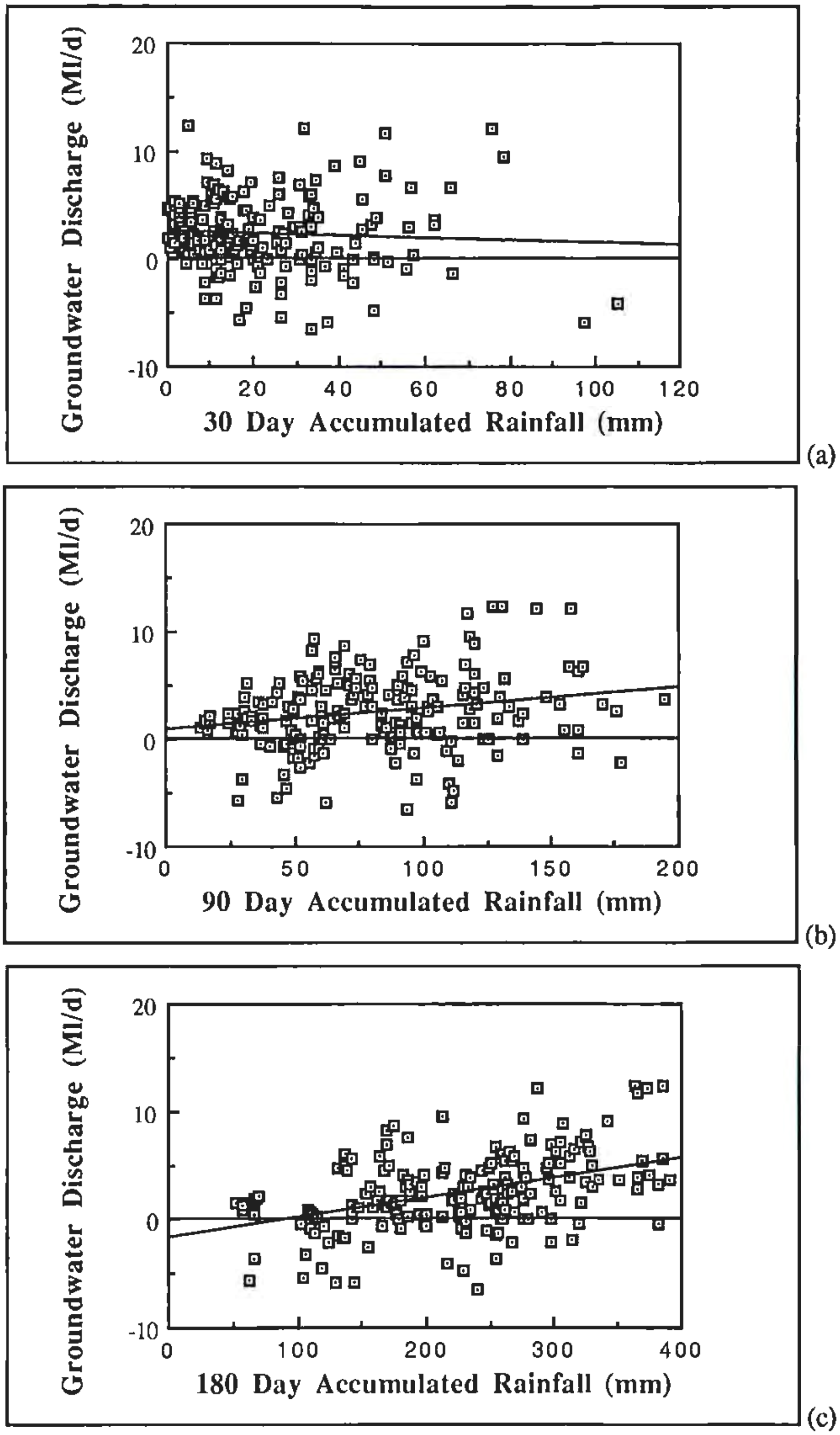


Figure 4.15: Relationship between groundwater discharge to the Wimmera River between Glynwylln and Glenorchy during low flow periods and the (a) 30 day, (b) 90 day and (c) 180 day antecedent rainfall. The regression lines shown have R^2 values of 0.00, 0.05 and 0.19 for the 30, 90, and 180 day antecedent rainfalls respectively.

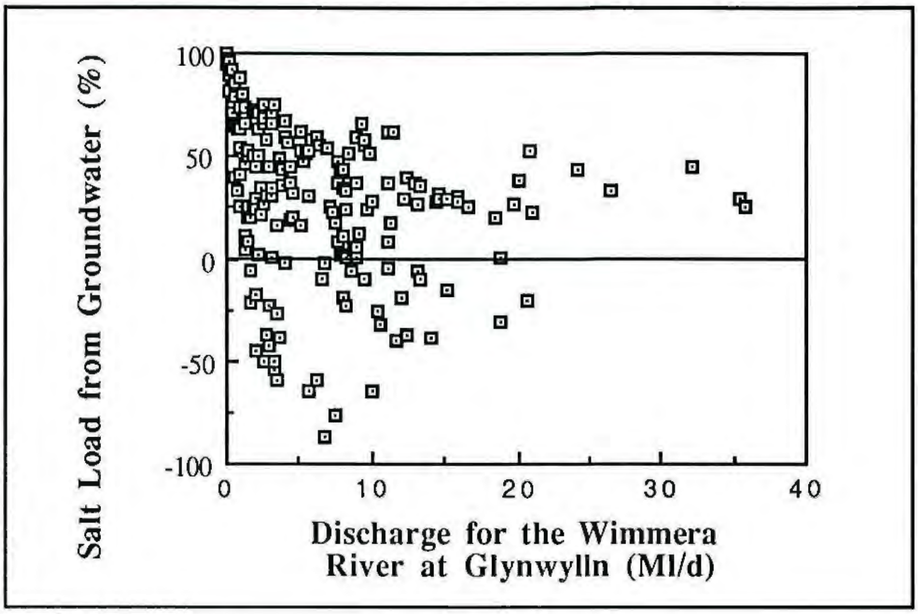


Figure 4.16: The relationship between the contribution of groundwater to the salt load entering the Wimmera River between Glynwylln and Glenorchy during low flow periods and the discharge in the Wimmera River at Glynwylln.

4.8.8 MODELLING IMPLICATIONS.

During low flow periods the salt load between Glynwylln and Glenorchy can be significantly affected by groundwater inflows. It is therefore useful to include the groundwater inputs into the model of the river so that low flow predictions of salinity will be more realistic. Although this analysis does not consider high flow periods, it would be expected that groundwater interaction would not have a significant impact on the salinity of the river during high flows because the groundwater flow is likely to be very small compared to the flow in the river. It is argued that groundwater inflows to the Wimmera River between Glynwylln and Glenorchy should be included in the model using a steady flow rate of 2.4 Ml/d which is based on the flow rate calculated in §4.8.3.1 using water balances during low flow periods. This will allow for the impact of groundwater discharge on the river salinity during low flow periods, but will not significantly affect predicted salinities during high flow periods. The groundwater inflows were included in the MIKE 11 model by specifying four lateral inflows of 0.6 Ml/d and 5 000 EC at chainages of 2.5 km, 7.5 km, 12.5 km and 17.5 km.

4.8.9 GLENORCHY TO HUDDLESTONS WEIR.

The only data available for calculating groundwater interaction with the Wimmera River between Glenorchy and Huddlestons Weir are discharge data. There are a number of ungauged tributaries which enter the river over this reach (Figure 4.17).

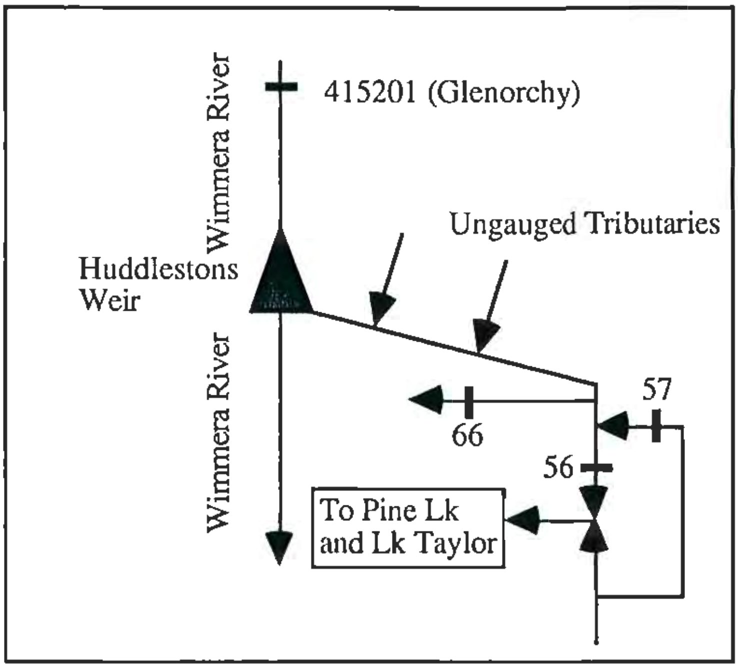


Figure 4.17: Inflows to and outflows from the Wimmera River and Wimmera Inlet Channel.

Another problem is that the diversion from the Wimmera River at Huddlestons Weir is only measured near Pine and Taylors Lakes (56). Inflows of water associated with the WMSDS may occur from both the Lubeck Loop Channel (57) and the Mt William Creek. Outflows occur in the Rocklands Lubeck Channel (66). To avoid problems with data errors from the WMSDS, flow balances were conducted for periods when only the River and Wimmera Inlet Channel were flowing. An implicit assumption was that all the river flow was diverted at Huddlestons Weir which is usually the case during low flow periods.

Figure 4.18 shows the groundwater flows. As can be seen the balancing item for this reach behaves erratically and is often an order of magnitude larger than is plausible (see §4.8.4.3). Given the lack of other sources of information and the problems with obtaining plausible groundwater flows from the available stream discharge data; the assumption that interaction between the Wimmera River and the groundwater for the reach between Glenorchy and Roseneath is insignificant will be made for the purposes of modelling the Wimmera River.

4.9 GROUNDWATER INFLOWS TO THE LOWER WIMMERA RIVER.

Inflows of groundwater occur along extensive reaches of the Wimmera River downstream of Roseneath (Anderson and Morison, 1989c; D. Strudwick, DCE, Per. Com.). The interaction between the groundwater and the Wimmera River

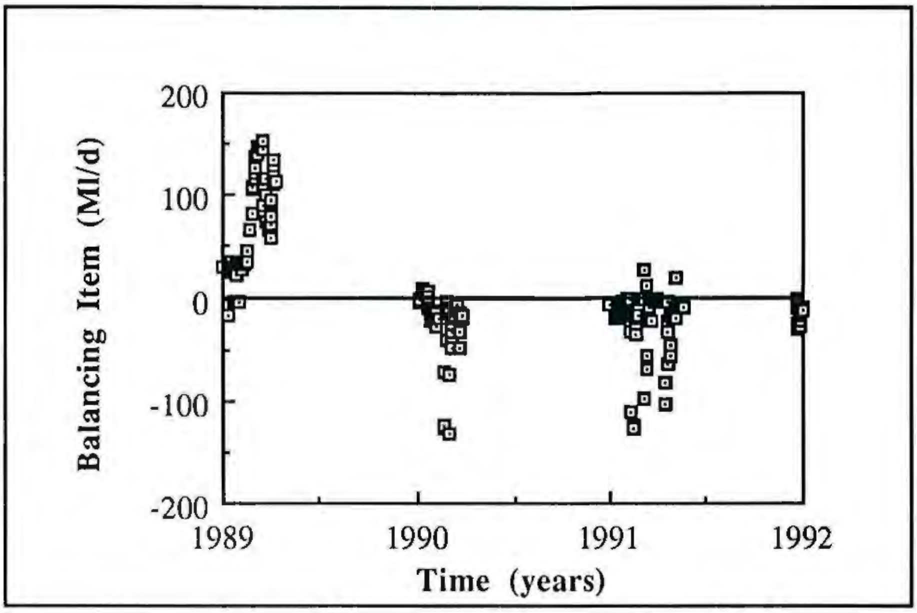


Figure 4.18: Time series of the balancing item for the Wimmera River and Wimmera Inlet Channel between Glenorchy and McKenzies Drop (56) for low flow periods.

downstream of Roseneath has been studied by the Victorian Department of Conservation and Environment (DCE), Centre for Land Protection Research (D. Strudwick, per. com.). However this work has not been published. Nevertheless estimates of groundwater inflow rates from the above study have been used in the MIKE 11 model of the Wimmera River primarily because resources were not available to repeat the DCE study. Table 4.6 is a summary of groundwater inflow rates provided by D. Strudwick (DCE, Per. Com.). Since it is not possible to review critically the DCE study these estimates have been accepted as provided and used to specify inflows of groundwater to the Wimmera River approximately every 1 km along reaches with significant groundwater inflows. Groundwater inflows were only included in the salinity simulations since groundwater flows are small compared with river flows but are highly saline.

4.10 SUMMARY.

The development of a data base of inflow discharges and salinities for the Wimmera River between Glynwylln and Lochiel has been described in this chapter. A significant number of ungauged tributaries enter the Wimmera River over this reach. Flows for these ungauged tributaries were estimated by scaling the Concongella Creek hydrograph by the ratio of the mean annual flow in the ungauged tributary to the mean annual flow in Concongella Creek. A regression relationship between catchment area and mean annual discharge was developed and

Chainage (km)	Discharge (Ml/a/km)	Salinity (EC)
84.0 - 84.5	248	20 000
117.0 - 125.0	50	11 000
125.0 - 145.0	40	7 000
145.0 - 148.5	220	33 000
148.5 - 156.0	10	40 000
156.0 - 157.0	220	33 000
160.2 - 161.0	40	40 000
162.3 - 162.7	50	25 000

Table 4.6: Estimates of groundwater inflow rates and salinities between Roseneath (84.0 km) and Lochiel (193.0 km) provided by Strudwick (DCE, Per. Com.). Reaches where there is no interaction between the groundwater and the Wimmera River and where the groundwater is being recharged from the river have been excluded from the table since they were not used in the MIKE 11 model.

used to obtain this ratio. Information available at flow gauges remote from the Wimmera River was combined with the above estimates where applicable.

Salinities for ungauged tributaries and the Wimmera River at Glynwylln also required estimation. Extensive use of solute rating curves was made for this estimation. A MVUE model was found to be most appropriate for estimating salinity at Glynwylln. For ungauged tributaries the index in the power curve describing the solute rating curve was assumed to be -0.3, which is typical of unregulated streams in the Wimmera River Catchment, and the coefficient was set such that an appropriate annual salt load resulted. Again information available at flow gauges remote from the Wimmera River was combined with the above estimates where applicable.

Inflows of groundwater to the Wimmera River between Glynwylln and Glenorchy were estimated to be 2.4 Ml/d. This estimate is used in the MIKE 11 model of the Wimmera River. Estimates of groundwater inflow rates to the Wimmera River downstream of Roseneath were obtained from a DCE study (D. Strudwick, DCE, Per. Com.) and are used in the MIKE 11 model.

CHAPTER 5 - RIVER GEOMETRY.

Physically based numerical models of river systems must be based on equations which describe the important flow phenomena found in those systems and on the real hydraulic and topographic features of those systems (Cunge et al., 1980). This chapter describes the way in which the geometric features of the Wimmera River system are incorporated in the MIKE 11 model of that system. Initially the geometric data requirements of a physically based river model are discussed. The geometric and hydraulic characteristics of natural alluvial channels are discussed in conjunction with the philosophy of stochastic modelling. A method of characterising individual cross-sections in the Wimmera River is then developed and used as the basis for a stochastic method of infilling cross-section data. A numerical experiment is then performed to assess the impact of large-scale channel variability in rivers like the Wimmera River.

5.1 TOPOGRAPHY AND HYDRAULIC MODELS.

Many authors have considered the issues relating to mathematical representation of flow processes in natural streams and the numerical solution schemes required to solve the resultant differential equations. On the other hand less attention has been given to the specification of river channel geometry. One of the exceptions and more detailed considerations is that of Cunge et al. (1980). Other relevant studies include work on the stochastic aspects of river geometry (Chiu, 1968; Chiu and Lee, 1971, 1973 and Chiu et al., 1976) and on the impact of channel variability on flow resistance (eg Miller and Wenzel, 1985; Hey, 1988; Shen et al., 1990 and Li et al., 1992). This section will discuss the geometric data requirements of a river model by drawing on the above work and some additional considerations. It does not consider the representation of artificial hydraulic features such as weirs which are generally geometrically well defined and therefore more easily incorporated in a model.

Cunge (1975) and Cunge et al. (1980) discuss the geometric representation of a natural river system in general terms. The topographic and geometric features of a river need to be represented so that the model correctly simulates the amount of water stored in each reach and so that the wave speed is unbiased (Cunge, 1975). To achieve this representative cross-sections are required at a spacing which will adequately describe the salient features of the system which are top width, cross-sectional area and conveyance as a function of river stage and river chainage

(Cunge et al., 1980). The variability of these features along the river is important.

The irregular bed slope and cross-sectional shape found in natural channels gives rise to irregular pressure and gravity forces in the flow direction. Thus the irregularities can be thought of as producing an irregular driving force (Chiu and Lee, 1971). Li et al. (1992) considered these irregular inputs from a stochastic perspective and, using a stochastic version of the steady state momentum equation, have shown that flow resistance in open channels is always increased by large-scale irregularity.

Ideally computational points should be chosen at all irregularities (Cunge,1975). However what constitutes an irregularity and therefore the required spacing depends on the purpose to which the model is to be put (Cunge 1975). On one hand, for a flood simulation model, storage may be dominated by the flood plain and conveyance by the channel. In such a case the valley width and mean channel characteristics as a function of stage may be all that is required and cross-sections can, subject to the requirement the water surface profile is defined adequately in space, be relatively widely spaced.

On the other hand, the hydraulic behaviour of a stream at low flows is dominated by topography of the channel; especially channel features, such as bars, that lead to backwater effects in the channel (Richards, 1982; Miller and Wenzel, 1985; Hey, 1988). The routing of a small flow event is dominated by the width of the channel at water level since this influences the transient storage; and by the conveyance of the controlling sections. Energy losses and therefore the water surface profile are also more strongly influenced by local losses at expansions and contractions than at high flows (Miller and Wenzel, 1985).

When water quality is to be simulated, the total storage in the channel, as opposed to the transient storage, is also important because this determines the residence time of water in the channel. Glover and Johnson (1974) have identified a lag effect which results from a difference in kinematic wave speed and mean water velocity. This displaces the pollutograph relative to the hydrograph and is a direct result of water being stored in the stream channel. The backwater effects due to topographic highs in the river bed can significantly affect this total storage, especially at low flows. As an indication, an approximate calculation of the water stored in the Wimmera River between Horsham and Lochiel can be made by assuming pools exist in 50% of this reach which is 90 km long. If the average pool width is 30 m

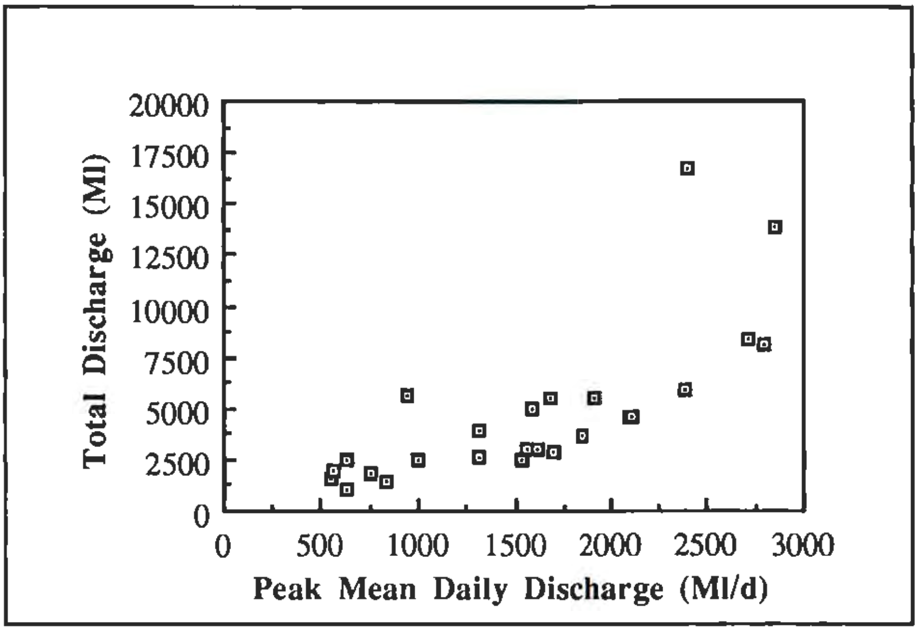


Figure 5.1: Total event discharge as a function of peak discharge for the Wimmera River at Horsham.

and the average depth is 2 m then the water stored is approximately 3 000 Ml.

Figure 5.1 demonstrates the relationship between peak mean daily flow and event discharge. Event discharge was defined as all mean daily flows above 100 Ml/d adjacent to the event peak except in the case of multiple peak events which were divided at flow minima provided the minima was significantly less than the smaller adjacent peak. Multiple peak events were typically large events which are of less interest in this study. It is apparent from this graph that a flow event peaking at 1 500 Ml/d will contain approximately the same amount of water as is stored in the channel at cease to flow level. Flows of 1 500 Ml/d are known to completely flush most of the saline pools in the Wimmera River (Chapter 8). Therefore the effect of channel storage due to topographic highs in the bed is likely to be significant during smaller events and low flow periods. Therefore this feature should be included in the specification of river channel geometry in the MIKE 11 model.

5.2 ALLUVIAL CHANNELS.

Alluvial river channels are naturally variable (Richards, 1982). This variability is the result of a number of interacting features of a river system including the imposed discharge from the catchment, the sediment load carried by the stream and its characteristics and the perimeter, or bed and bank, sediments that form the channel boundaries (Leopold et al., 1964; Richards, 1982). There are systematic components which can be identified in this variability and there is also a random component. The variation of natural channels occurs over a number of scales.

At the catchment-scale there is a tendency for width, depth and therefore area to increase downstream and for the width to increase more quickly than the depth (Leopold et al., 1964). Since the width increases more quickly than the depth there is a scale dependence in the cross-sectional shape (Richards, 1982). There is also a tendency for the channel slope to decrease downstream which leads to a concave up long profile (Leopold et al., 1964; Richards, 1982).

Another systematic variation that can be identified in rivers is the meander length, λ_m. There is a marked tendency for river channels to curve back and forth as they progress down a valley; indeed straight sections of natural channel are rare (Langbein and Leopold, 1966). Meandering is often closely tied to the pool-riffle sequence since there is generally a pool-riffle-pool-riffle sequence associated with each meander sequence or a pool on each bend with a riffle between each bend (Richards, 1982). The pool riffle length is usually approximately 5-7 times the mean channel width (Leopold et al., 1964; Richards, 1982). Riffles are characterised as topographic high points in an undulating bed long profile which have rapid shallow flow and a steep water surface gradient (Richards, 1982).

Although this length has been identified as typical there is a great deal of variability in the spacing of pools and riffles in a typical river (Richards, 1982). This is demonstrated by Richards (1976) who studied the characteristics of a number of bed elevation series measured at a scale significantly shorter than the riffle-pool-scale. Using harmonic analysis, autoregressive models and spectral analysis Richards identified a pseudo-cyclic behaviour that could be explained with varying success (R^2 = 0.41- 0.98) by a second order autoregressive model. The harmonic and spectral analyses showed that a variety of length-scales were involved in the variation. The analyses showed that the average wavelength was approximately 5.6 times the channel width which is consistent with other observations of pool-riffle spacing.

Although there are systematic effects apparent in river channel variation, there is also a significant degree of variability introduced by the effect of random influences on the system. This variability raises the possibility of stochastic representation of a river channel. Chiu and various co-authors have developed a methodology for

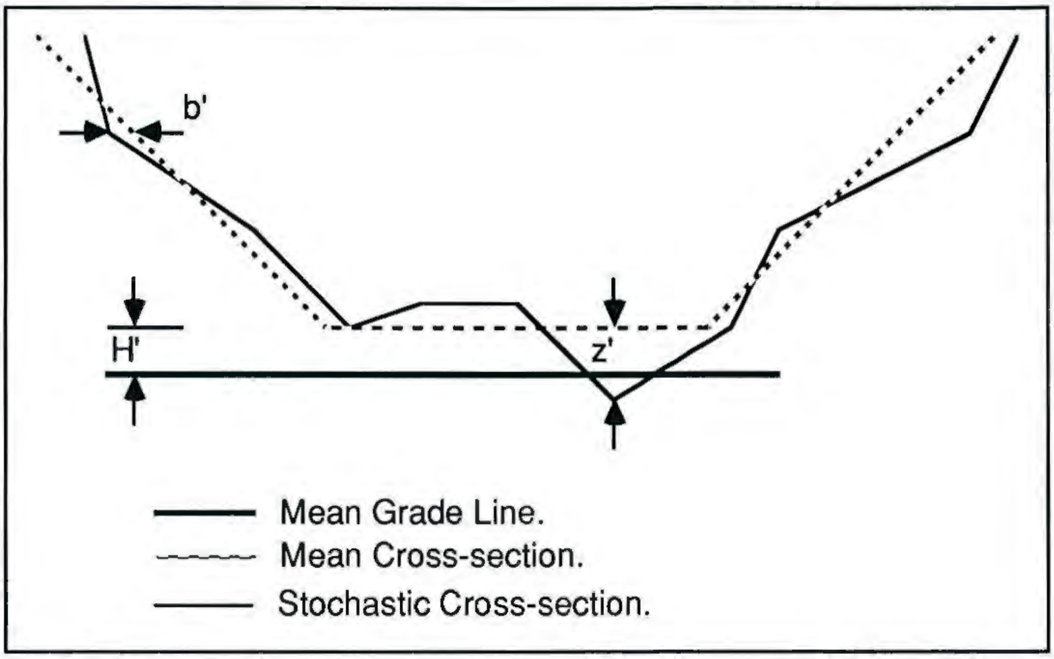


Figure 5.2: Chiu and Lee's (1971) stochastic cross-section model.

generating stochastic river cross-sections (Chiu, 1968; Chiu and Lee, 1971, 1973; Chiu et al 1976). This method generates a cross-section by perturbing an assumed average cross-section. The average cross-section can be scaled according to the location along the river. The mean invert is calculated from a trending invert and a random vertical displacement. Perturbations are then introduced into the cross-section shape by vertically displacing the bed at intervals across the bed and by horizontally displacing the banks at specific vertical intervals (Figure 5.2). Small-scale irregularities (ie. characterised by a length-scale significantly smaller than the channel width) in the shape of the cross-section are included in this model.

The displacements are treated as spatial series in the following manner. The perturbations to the width, b', form a one-dimensional spatial series which contains the perturbation from the corresponding point on each cross-section. The two sides of the cross-section are treated separately. In other words the perturbations second up from the bottom on the left side of each cross-section form one series and so on. The displacement of the mean invert, H', from the trending invert is treated similarly. The displacement of the bed, z', from the mean invert is treated as a spatial series across the bed of each cross-section.

These series are treated as random walks and two models were considered for the walk increments. The first model was a truncated Brownian motion model in which each increment was independent and normally distributed and the second model was a first-order autoregressive model. The simulation interval used was determined by finding the interval at which serial correlation in the increments appeared (ie. the interval is decreased from some long interval until serial correlation in the increments appeared) and the increment variation was treated as a linear function of the simulation interval. It was found that both models were adequate, but that the first-order autoregressive model led to a more precise simulation of the water surface profile (Chiu and Lee, 1971).

Chiu and Lee (1971) generated cross-sections with small-scale variations from an assumed mean shape. An alternative approach is to smooth over the small-scale variations in some way and to characterise the shape of the smoothed cross-section. The variation in cross-section-scale channel characteristics along the stream is retained and can be analysed. The following section provides such an analysis of available cross-sections for the Wimmera River.

5.3 CHARACTERISING THE WIMMERA RIVER CHANNEL.

In the lower reaches of the Wimmera River the river channel is very variable. The river channel alternates between areas in which there are multiple channels, typically two, which have low banks and small cross-sectional areas at bank-full discharge (wetland); areas which are typified by sequences of shallow pools and runs (channel); and areas in which large volumes of water are stored in deep pools (large pool). The term run is used in preference to riffle, due to the slow nature of flow over the topographic high points between meander bends in the Wimmera River. The pool-run sequence in the Wimmera River is analogous to a pool-riffle sequence in that deep points are located on bends and shallow points between bends. The different channel types can be observed in the field and can also be delineated from aerial photographs and, to a lesser degree, topographic maps. Figure 5.3 shows a typical aerial photograph of the Wimmera River with examples of each channel category marked. Since there are differences in the channel of the Wimmera River that can be observed in the field, the cross-sectional characteristics of the channel for each channel type were examined using available cross-sectional data. Cross-sectional data are available for three reaches of the Wimmera River (Figure 5.4) which are referred to as reach 1, reach 2 and reach 3. The locations of measured cross-sections were examined on aerial photographs in an attempt to identify any biases in the sampling. No biases were evident.

Figure 5.3: Aerial Photograph of the Wimmera River showing examples of the different channel categories. A-large pool, B-channel, C-wetland.

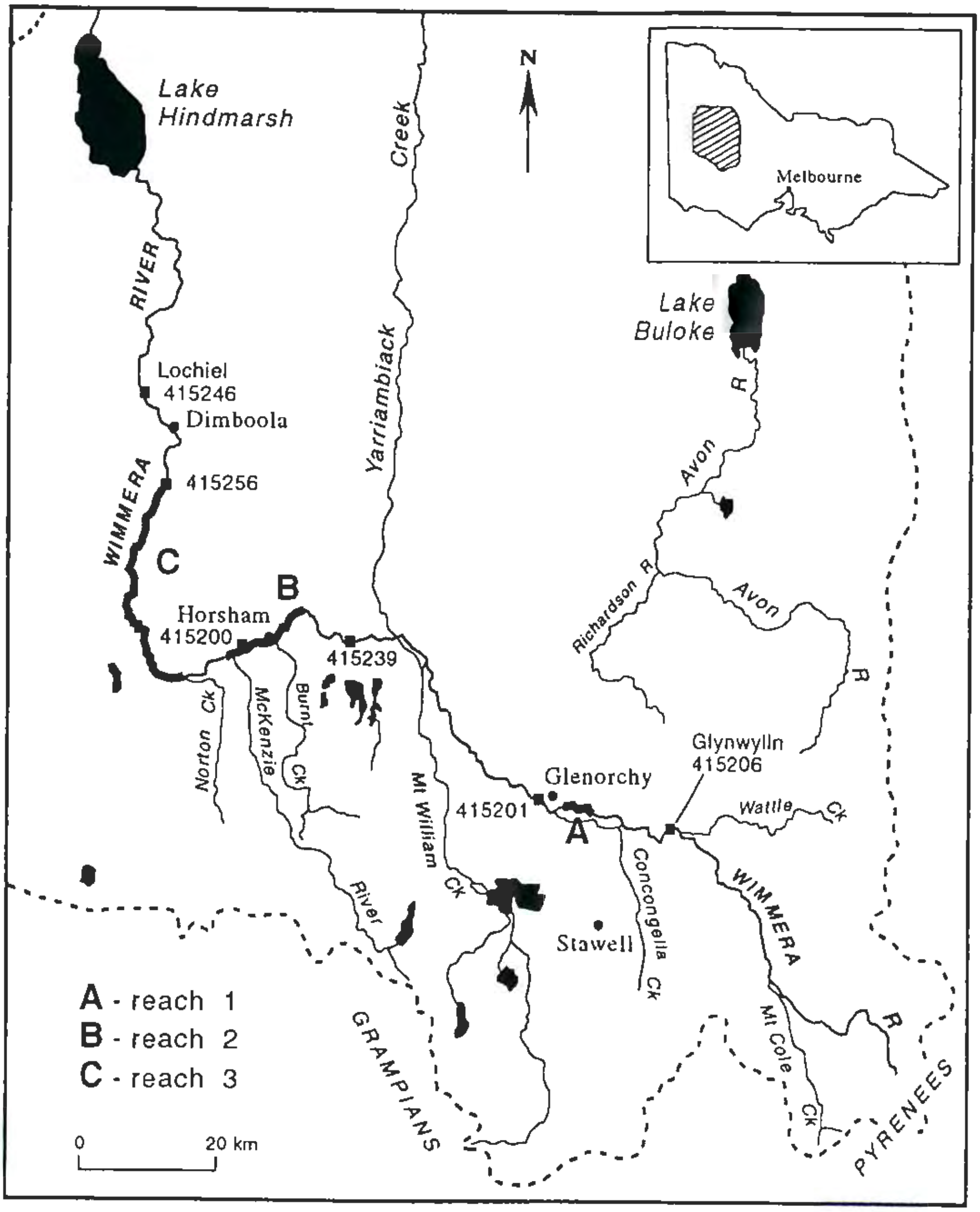


Figure 5.4: Reaches of the Wimmera River from which cross-sectional data has been used.

The following methodology was used. Each cross-section was allocated to one of three classes: wetland, channel and large pool. The cross-sections representing channel were subsequently divided into two subclasses since the available cross-sectional data comes from two areas. The first, called upstream channel (reach 1), is near Glenorchy (Figure 5.4) and the second, simply called channel, extends from upstream of Horsham to Big Bend (reaches 2 and 3) (Figure 5.4). While the second group of data comes from an extensive area most of the major flows that occur in that area are generated upstream of it and the mean channel slope is also similar throughout the area. For these reasons the channel characteristics were assumed constant for each channel type throughout the second area.

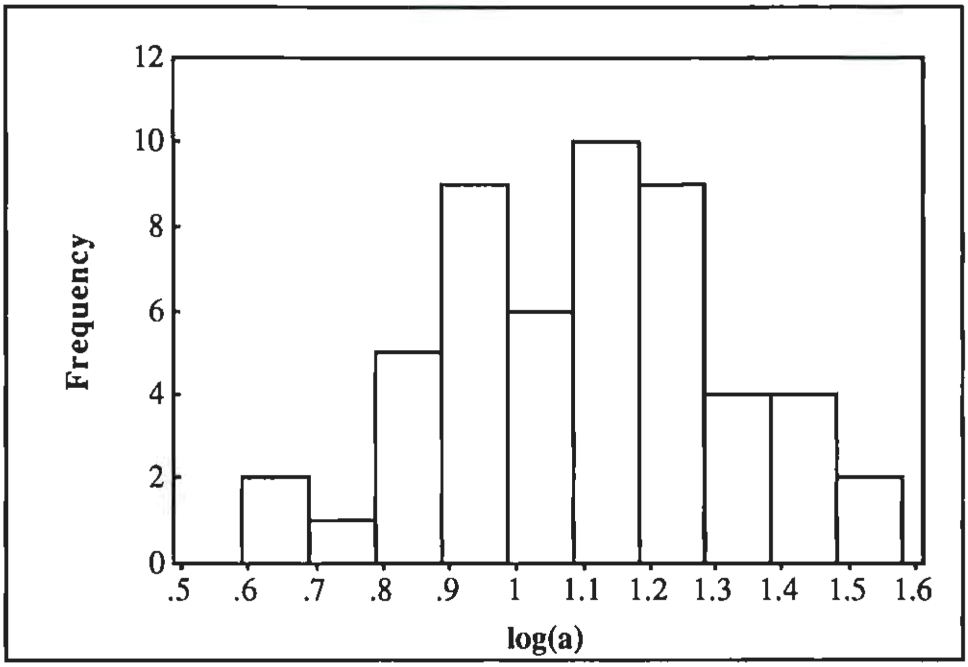
After the cross-sections had been classified, it was necessary to characterise each individual cross-sections. One possible way to characterise the cross-section shape for the purpose of calculating the hydraulic parameters for 1-dimensional flow is simply to find a relationship between the width and the depth of a cross-section. Garbretch (1990) used power curves to represent this relationship. This approach is simple but, in some cases, has the disadvantage poorly representing highly irregular and compound channels (Cunge et al., 1980). Since the in-bank characteristics of cross-sections are of most interest in this study the compound channel problem is not encountered and a power curve (Equation 5.1) was fitted to each available cross-section.

$$W = a Z^{b}. \tag{5.1}$$

In Equation 5.1 W is the cross-section width at elevation Z above the invert and a and b are the fitted coefficient and index respectively.

Coefficients of determination for the fit of Equation 5.1 were all in excess of 0.85 and 0.6, and typically greater than 0.95 and 0.85, in the logarithmic and real domains respectively. The quality of the fits for each cross-section indicates that Equation 5.1 is an adequate model of the variation of cross-section width with depth for the Wimmera River. However it should be noted that for some cross-sections Equation 5.1 under-predicts the width for elevations close to the top of the channel banks. This is partly due to the top of the bank being poorly defined at some cross-sections.

It is also necessary to characterise the vertical position and vertical extent of each cross-section. The vertical position was characterised by fitting a linear regression of invert elevation as a function of chainage to sections of the channel with constant grades and taking the residual as a measure of displacement from the mean channel grade-line. This method has been used in the past by Chiu and Lee (1971) and by Richards (1976). The bank-full depth, Z_{bf}, was used as a measure of the vertical extent. Top of bank elevations were obtained by examining plots of the available cross-sections. Where bank heights on either side of the cross-section varied significantly, the lower value was used. Figure 5.5 provides histograms of the data set and Figure 5.6 shows the variation of a, b and r along the channel. Table 5.1 summarises the statistical characteristics of a, b and r.



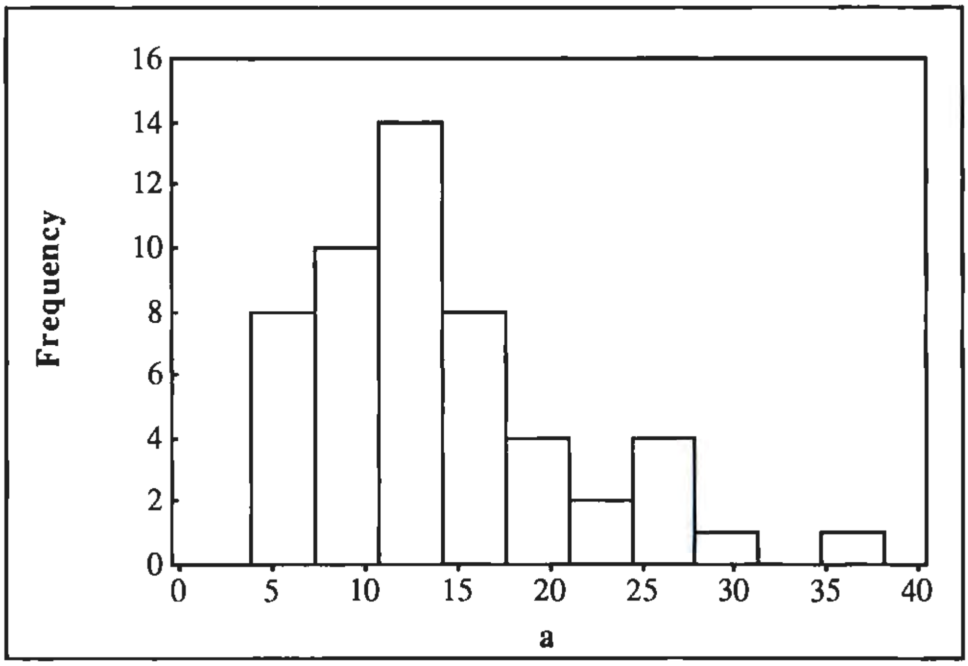
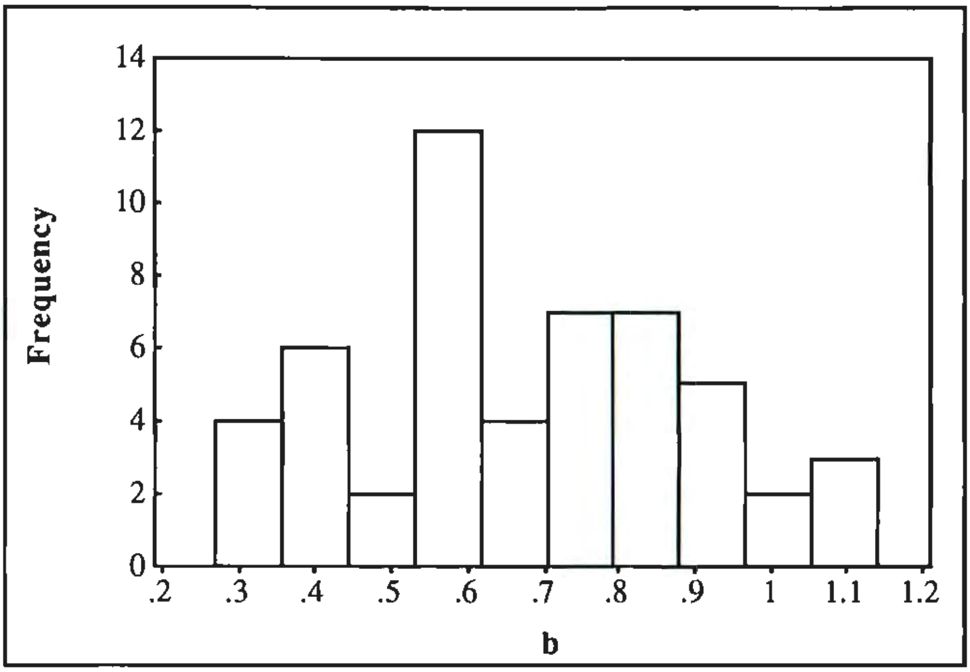


Figure 5.5: The frequency distribution of each cross-section parameter for the entire data set.



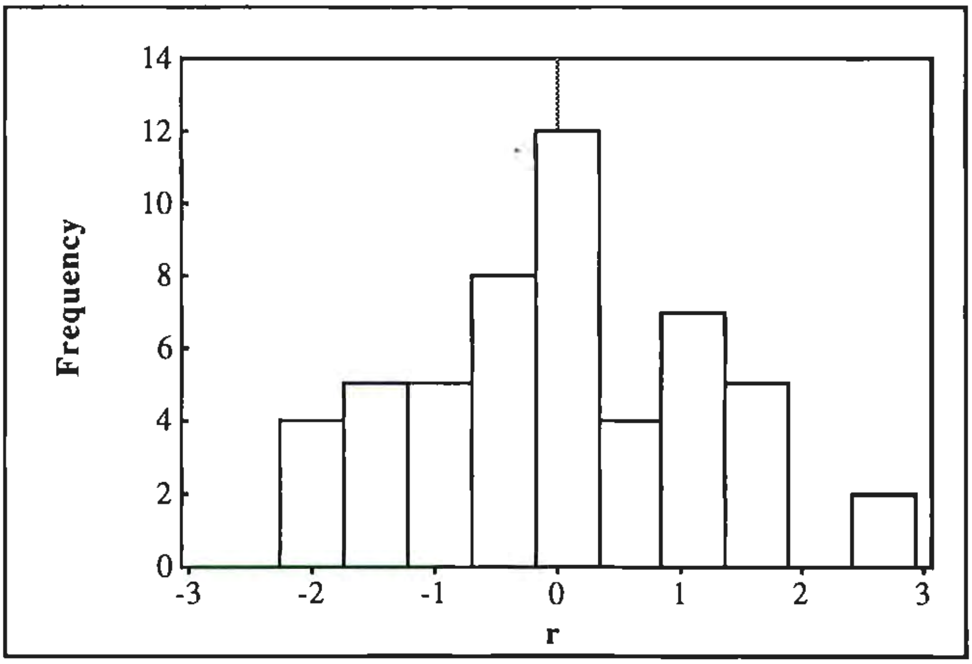


Figure 5.5 (cont): The frequency distribution of each cross-section parameter for the entire data set.

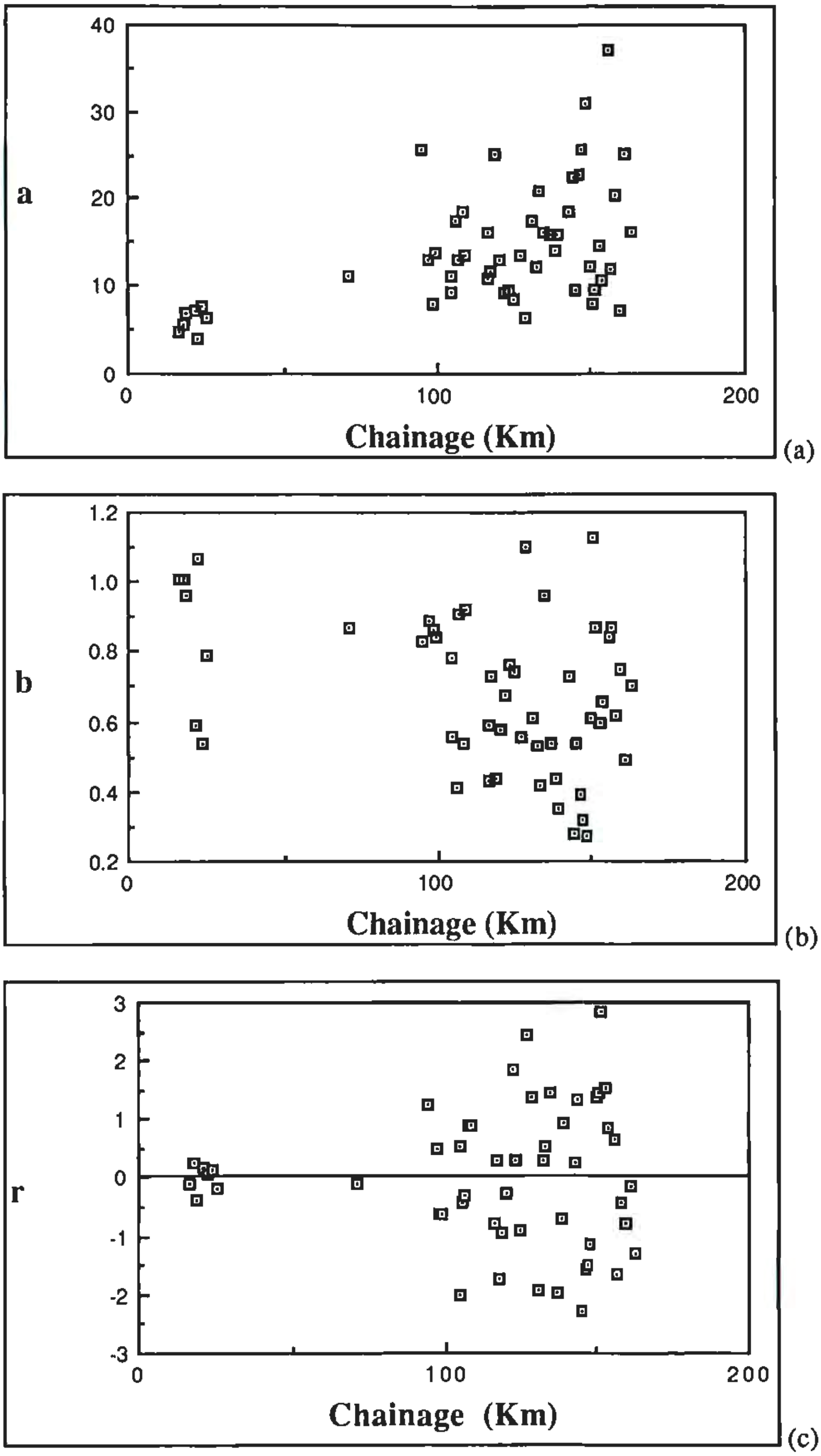


Figure 5.6: Variation of cross-section parameters along the Wimmera River. Chainage increases downstream from Glynwylln Gauging station.

Parameter	log(a)	a	b	r	Z_{bf}
Mean	1.10	14.17	0.68	0	5.86
Standard Deviation	0.21	6.98	0.22	1.18	2.01
Coefficient of Variation	0.19	0.49	0.32	na	0.34

Table 5.1: Mean, standard deviation and coefficient of variation of cross-section characteristics for the Wimmera River.

The above analysis results in three parameters describing each cross-section. The variables required to describe the hydraulic properties of a cross-section are the width, area and hydraulic radius. Width is obtained directly from the fitted relationship and cross-sectional area can be calculated by integrating over the depth of the flow. The hydraulic radius can be calculated from the cross-sectional area and the wetted perimeter. If the cross-section is assumed to be symmetric, the wetted perimeter, P, at a depth, y, can be calculated as:

$$P = 2\int_0^y \sqrt{\left(\frac{abD^{b-1}}{2}\right)^2 + 1}\quad dZ\,. \qquad (5.2)$$

Equation 5.2 was solved numerically when calculating hydraulic parameters for the MIKE 11 model. Although Equation 5.2 assumes a symmetric cross-section, the value of P calculated is insensitive to this assumption. Since most cross-sections in the Wimmera River are nearly symmetric, use of Equation 5.2 is adequate.

5.3.1 INTERPRETATION OF THE CROSS-SECTION PARAMETERS.

The basic parameters fitted to the cross-section are a, b, r and Z_{bf}. a and b represent a smoothed relationship between width and elevation, r characterises the relative vertical position and Z_{bf} characterises the vertical extent of the cross-section. It is possible to non-dimensionalise Equation 5.1 using the bank-full depth, Z_{bf} as the scale parameter.

$$W^* = a^* Z^{*b}\,. \qquad (5.3)$$

In Equation 5.3, the non-dimensional width is $W^*=W/Z_{bf}$, the non-dimensional depth is $Z^* = A/Z_{bf}$ and $a^*=a/Z_{bf}^{1-b}$. a* is the bank full width to depth ratio for that

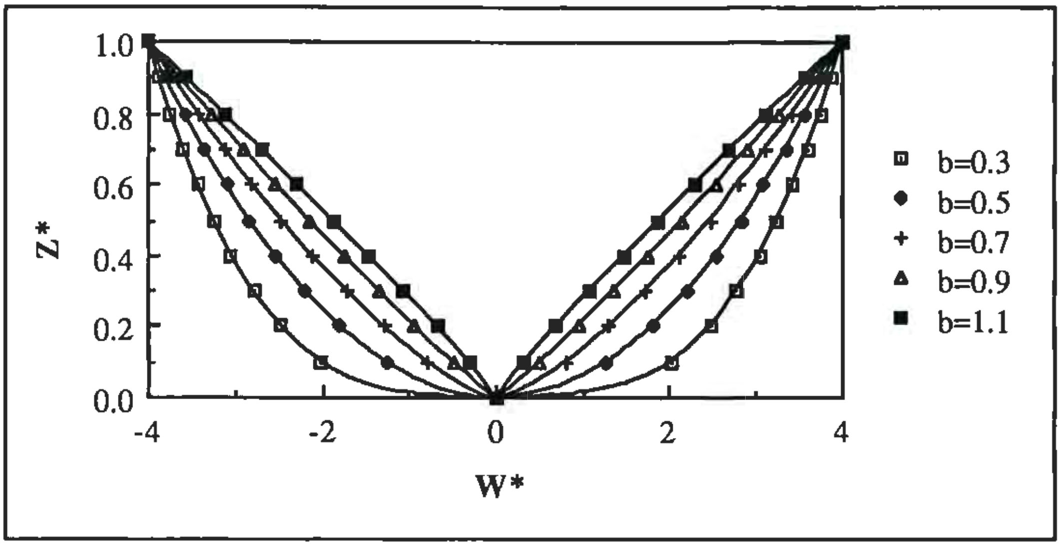


Figure 5.7: Variation in cross-section shape with b for a cross-section with a bank-full width to depth ratio equal to 8. Z* is the non-dimensional depth and W* is the non-dimensional width.

cross-section and b, which is unaffected by the non-dimensionalisation, represents the cross-sectional shape. Figure 5.7 shows the variation in cross-sectional shape with b assuming a symmetric cross-section. Variations in a and b between cross-sections provide a measure of the longitudinal variation in cross-sectional shape.

The vertical displacement of the cross-section from the mean grade line is represented by r. Because of the role of topographic high points in inducing backwater effects; the standard deviation of r is an important determinant of pool depths during low flow periods. Other cross-sectional properties of interest such as the bank-full cross-sectional area, A_{bf}, can be calculated from the above variables.

5.3.2 THE CHARACTERISTICS OF a, b AND r.

There are a number of ways of examining the data sets of a, b and r. The following section discusses the type of distribution of the entire data set, differences between the categories, the variation of the cross-sectional properties along the stream, cross-correlation between a, b and r and the spatial-correlation of the data sets.

5.3.2.1 Distribution.

An examination of Figure 5.5 indicates that the distributions of b and r are symmetric and close to normal. The distribution of a is skewed right; however the distribution of log(a) is symmetric and close to normal. The hypotheses that a is lognormally distributed and that b and r are normally distributed were tested using a Chi-Squared test which suggested that there was no significant difference (5% significance level) between the samples and hypothesised distributions. Further analyses assume that a is lognormally distributed and that b and r are normally distributed.

The categorised data were examined to determine if the categories were statistically different. Three separate hypothesis tests were conducted. These were, that the wetland category was different from the channel category, that the channel category was different from the large pool category, and that the channel category was different from the upstream channel category. The first two of these were chosen because they test for difference between the most similar categories in one locality, while the third was chosen to test for differences between the same category in two locations.

Hotelling's T test (Harris, 1985) was used to test these hypotheses. Hotelling's T test is based on the following equations.

$$F = (n_1+n_2-p-1)\, T^2 / (p\,(n_1+n_2-2)) \tag{5.4}$$

$$T^2 = \frac{\left| S_c + n_1 n_2 (\overline{\mathbf{X}_1} - \overline{\mathbf{X}_2})(\overline{\mathbf{X}_1} - \overline{\mathbf{X}_2})^T / (n_1+n_2) \right|}{\left| \mathbf{S}_c \right|} - 1 \tag{5.5}$$

$$S_c = \frac{A_1+A_2}{n_1+n_2-2} \tag{5.6}$$

$$\mathbf{A} = \mathbf{X'X'}^T \tag{5.7}$$

Where F is distributed $F_{(p,\, n1\, +\, n2\, -\, 2)}$, n_1 and n_2 are the number of samples in the two data sets, p is the number of parameters characterising each point (cross-section in this case), $\overline{\mathbf{X}}$ is a p x 1 vector of the means of the respective data sets and **X'** is an n x p matrix of deviations from the means of the respective data sets. Statistical differences (5% significance level) exist between the wetland and channel categories and between the channel category and the upstream channel category but not between the channel category and the large pool category.

The channel and large pool categories were proposed on the basis of apparently observable differences in the river channel related to the size of pools during the low flow periods. While there may be differences between the channel and large pool categories in the size of pools, this may be related to the influence of different downstream controls causing more or less pronounced backwater effects, rather

	Wetland			Downstream Channel			Upstream Channel		
Parameter	μ	σ	cv	μ	σ	cv	μ	σ	cv
log(a)	1.02	0.16	0.16	1.20	0.16	0.13	0.77	0.11	0.14
b	0.75	0.20	0.27	0.62	0.21	0.34	0.85	0.22	0.25
r	1.02	1.19	na	-0.37	1.09	na	0	0.22	na
Z_{bf}	4.22	1.53	0.36	5.72	1.39	0.24	9.36	0.23	0.02

Table 5.2: Mean, standard deviation and coefficient of variation of cross-section parameters for different categories of the Wimmera River Channel.

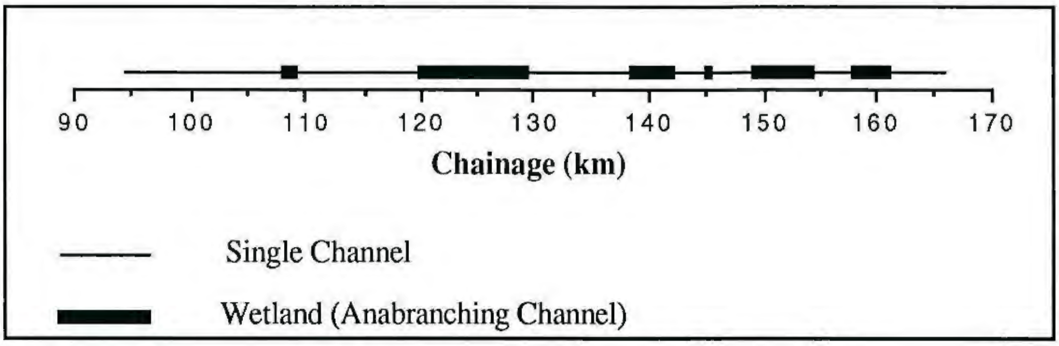


Figure 5.8: The distribution of channel and wetland categories for reach 2 and reach 3.

than to differences in the river channel *per se*. Therefore these two categories were combined to form a downstream channel category. The statistics of log(a), b and r for each category are presented in Table 5.2. When the channel and large pool categories are combined, the task of categorising the river channel becomes one of identifying anabranching sections which are categorised as wetlands. The distribution of the channel and wetland categories at and downstream of Horsham is shown in Figure 5.8. It should be noted that the channel and wetland categories have a length-scale that is significantly longer than the meander length-scale and that systematic variations in channel properties are associated with this larger scale pattern.

5.3.2.2 Longitudinal Trends.

The data set was examined in an attempt to identify differences between the cross-sectional characteristics adjacent to Glenorchy (reach 1), adjacent to Horsham (reach 2) and downstream of Horsham (reach 3). Hotelling's T tests were performed on the data sets for neighbouring areas. These tests identified a statistically significant difference between the cross-sections adjacent to Glenorchy

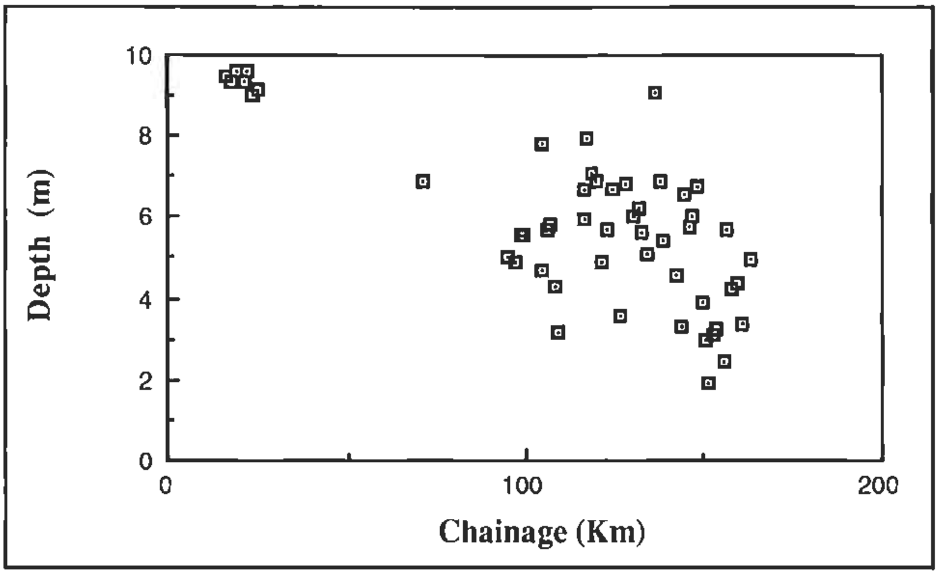


Figure 5.9: The variation of channel depth with chainage. Chainage increases downstream from Glynwylln Gauging station.

and those adjacent to Horsham, but not between those adjacent to Horsham and those downstream of Horsham. The difference between cross-sections at Glenorchy and those at and downstream of Horsham is evident in Figure 5.6.

There is a tendency for the invert to become more variable downstream (Figure 5.6c). This increasing variability means that the depth of pools under low flow conditions increase downstream. Figure 5.9 shows the variation in channel depth along the Wimmera River. There is a tendency for the average channel depth to decrease downstream which is contrary to what is often observed (Richards, 1982). Figure 5.10 shows that the bank full width to depth ratio, a*, for the Wimmera River at and downstream of Horsham is greater than that near Glenorchy. There is a slight downstream decrease in b (Figure 5.6b) which indicates that cross-sections become more U and less V shaped (Figure 5.7) further downstream.

Bank-full cross-sectional areas, A_{bf}, were calculated for each cross-section using:

$$A_{bf} = \frac{a}{b+1} Z_{bf}^{b+1} \quad . \tag{5.8}$$

Figure 5.11 indicates that there may be a slight downstream decrease in bank-full cross-sectional area; however a linear regression relationship between A_{bf} and chainage is not statistically significant (F test, 5% significance level). Therefore the observed downstream decrease in depth is compensated for by an increase in

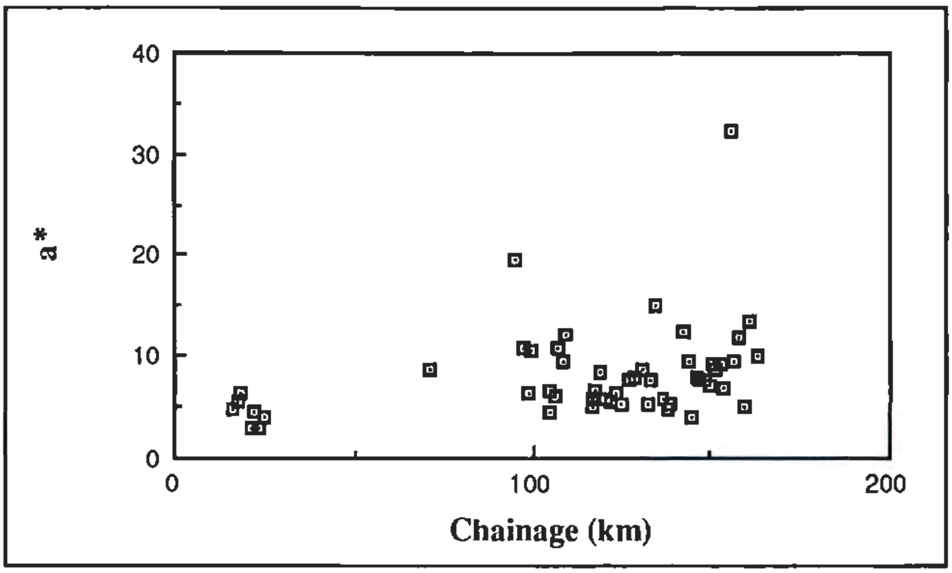


Figure 5.10: Variation of width to depth ratio along the Wimmera River. Chainage increases downstream from Glynwylln gauging station.

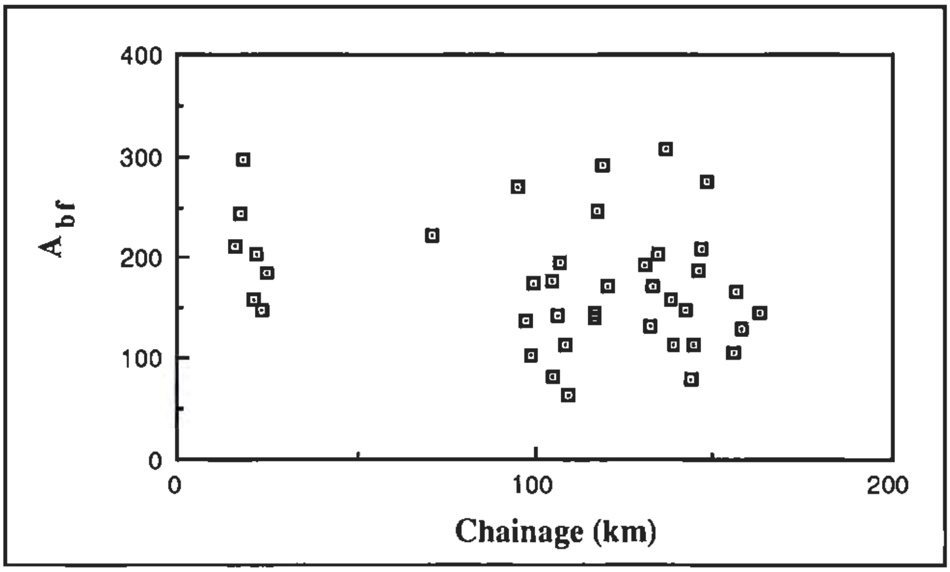


Figure 5.11: Variation in bank-full cross sectional area along the Wimmera River. Chainage increases downstream from Glynwylln gauging station.

channel width. Table 5.3 provides a summary of the above trends. It should be noted that cross-sections from wetland areas were excluded from the calculation of channel characteristics provided in Table 5.3 so that channel anabranching would not affect the calculated values.

5.3.2.3 Cross-Correlation.

The null hypothesis that the correlation is zero can be tested using the t statistic,

Locality	μ_a	μ_b	σ_r	$\bar{Z}$	$\bar{a}^*$	$\bar{A}_{bf}$
Glenorchy (Reach 1)	6.04	0.85	0.22	9.36	4.4	207
Horsham (Reach 2)	14.0	0.77	0.94	5.40	9.6	153
Downstream of Horsham (Reach 3)	18.4	0.55	1.13	5.88	8.9	175

Table 5.3 Changes in mean channel characteristics for the channel category along the Wimmera River.

with n - 2 degrees of freedom, defined by Equation 5.9. This test assumes the data are from a bivariate normal distribution (Yamane, 1973).

$$t = \frac{r}{\left(\frac{1-r^2}{n-2}\right)^{0.5}} \qquad (5.9)$$

In Equation 5.9, n is the sample size and r is the correlation coefficient. For the whole data set there is a statistically significant ($r = -0.604$, $t = -5.36$, $n = 52$) correlation between log(a) and b. The correlations between log(a) and r, and between b and r are not significant at a 5% significance level. Figure 5.12 shows the relationships between log(a), b and r for the entire data set and Figure 5.13 shows the relationships for each of the three categories.

When the data set is divided into the three categories the correlation between log(a) and b is still significant and the correlations between log(a) and r, and b and r are generally lower than between log(a) and b (Figure 5.14). All the correlations for the Upstream Channel are much higher and are statistically significant. This indicates that, in addition to the upstream channel being less variable than the channel in the lower reaches of the river, the variability in the cross-sectional shape and vertical position are more systematically organised.

The reason that the correlations between log(a) and b are high for both the categorised data and for the entire data set is that, although the categories tend to occupy different regions of the {log(a), b} space, they tend to all fall on the same line. Wetland and upstream channel categories are likely to be found above and left of the downstream channel category. This indicates that the wetland and upstream channel sections are more typically triangular and that the downstream channel sections become more concave. The negative correlation indicates that there is a tendency for the impact of changes in a and b on channel width at bank full discharges to be in opposite directions. This suggests that part of the variability in a and b would not be evident in the variability of the width to depth ratio.

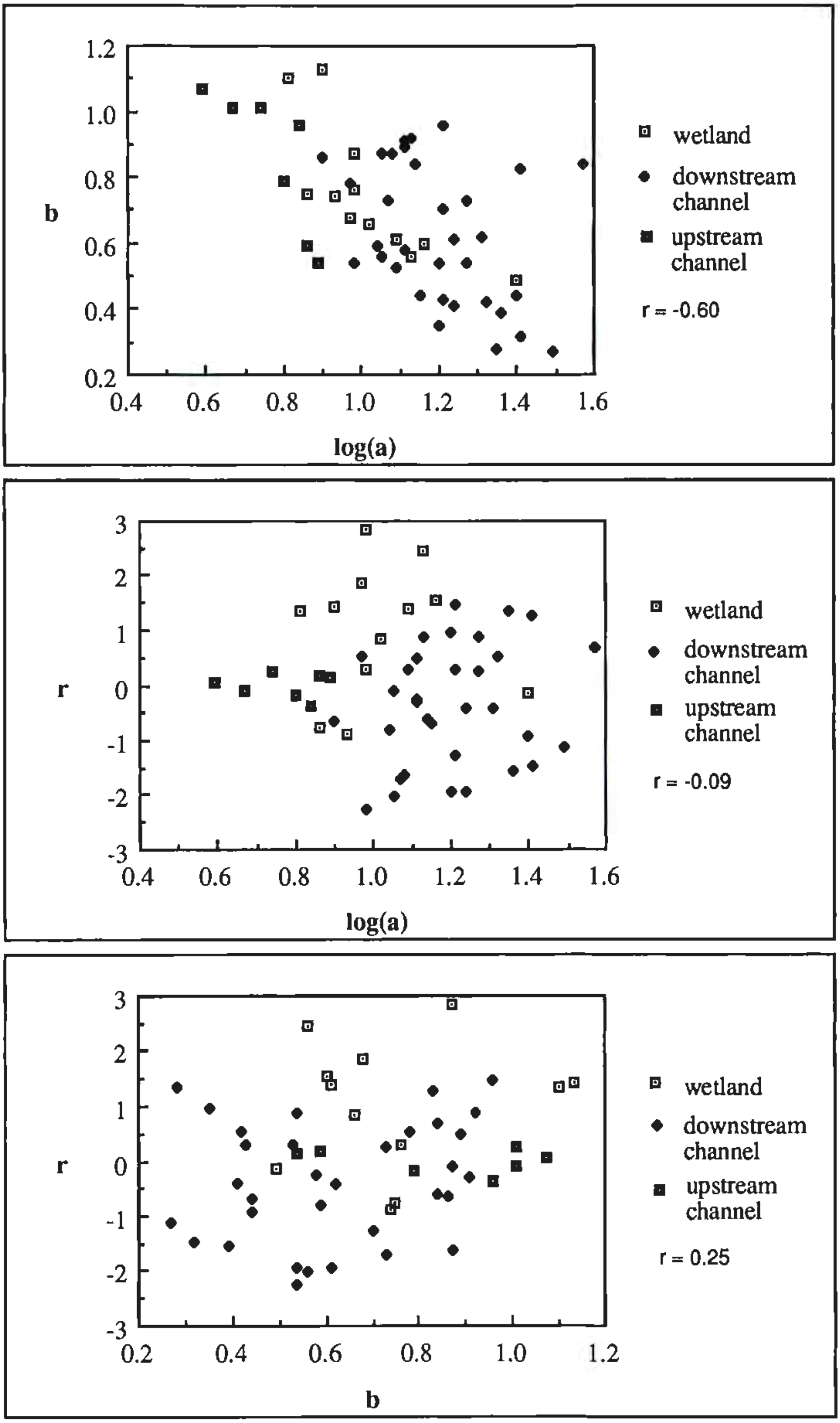


Figure 5.12 Correlations between cross-section parameters log(a), b and r for the Wimmera River.

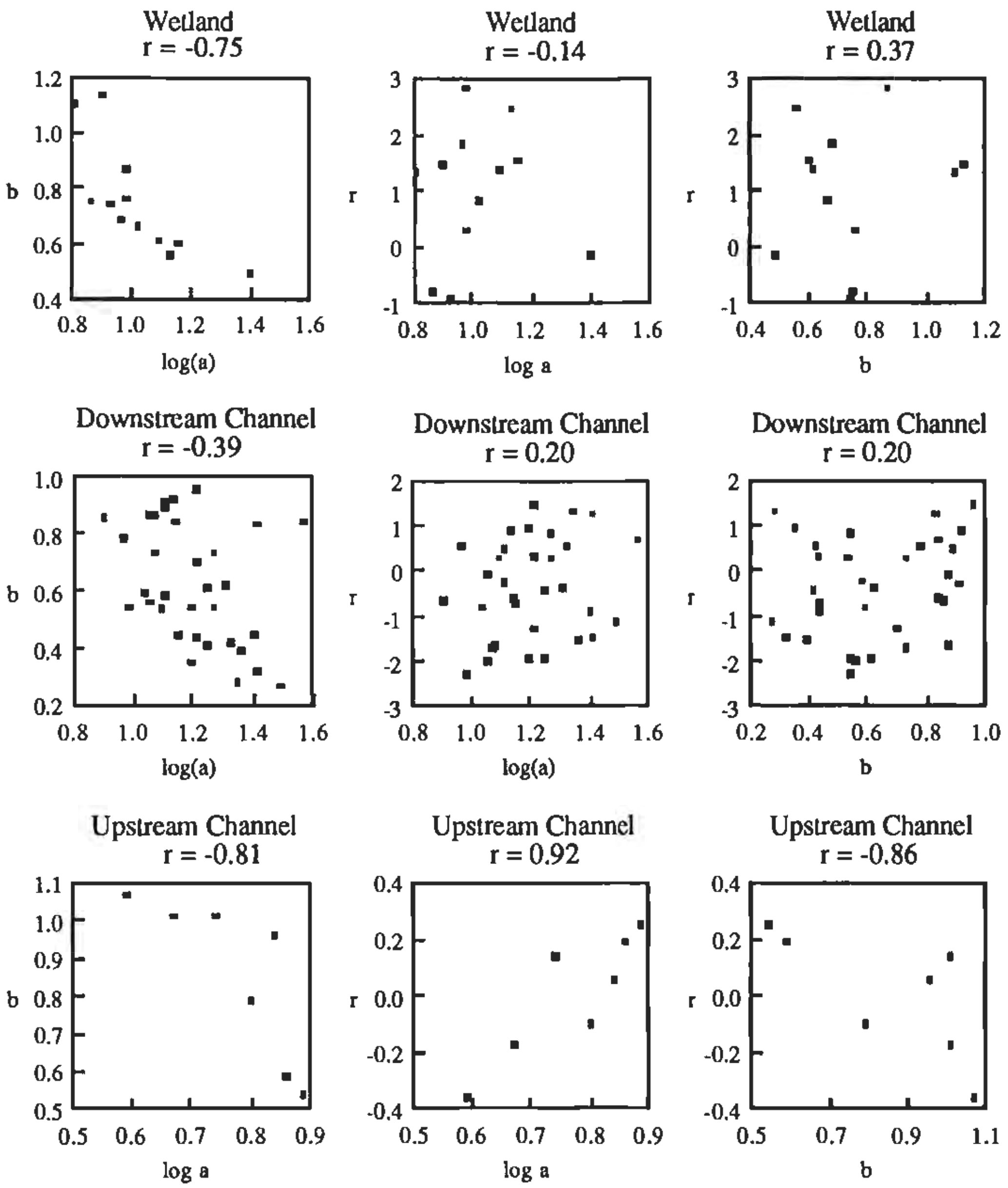


Figure 5.13: Correlations between cross-section parameters log(a), b and r for each channel category of the Wimmera River.

Two different sets of relationships between log(a) and b were examined to test the tendency for log(a) and b to lie on the same line. The first relationship consisted of a linear regression using the entire data set and the second consisted of three linear regressions - one for each category. A partial F test was used to test the difference between these two models which was insignificant at the 5% significance level. The relationship based on the entire data set is the more appropriate due to its relative simplicity.

5.3.2.4 Spatial Variability.

The variability of cross-sectional properties as a function of spatial separation was examined by constructing semi-variograms for each parameter for the cross-sections at and downstream of Horsham. The semi-variogram function is (Isaaks and Srivastava, 1989):

$$\gamma(s) = \frac{1}{2n}\sum_{i=0}^{n}(x_i-y_i)^2 \tag{5.10}$$

In Equation 5.10, γ is the value of the semi-variogram function, s is the spatial separation, n is the number of pairs used to calculate γ, and x and y are the data values corresponding to each pair. Semi-variograms were constructed as follows. Pairs of cross-sections at and downstream of Horsham (reaches 2 and 3) which had an appropriate separation were selected. Because of the irregular spacing of cross-sections it was necessary to allow a tolerance when selecting cross-section pairs. γ was calculated for separations of 1, 2, 3, ... 50 km and a tolerance of ±0.5 km was allowed. Distances were measured along the river channel. Figure 5.14 shows the resulting semi-variograms which are slightly different to those usually constructed for a spatial data set due to the use of a curvilinear coordinate system aligned to the river channel.

Figure 5.14 indicates that spatial variability of a, b and r is approximately independent of separation for the scales considered. The smallest separation in the semi-variograms is 1 km which is too large to capture spatial variation at the pool-run-scale in the Wimmera River. It may therefore be expected that the spatial variability may not depend on separation for the scales considered above. However a careful examination of Figure 5.14c shows that there tends to be a quasi-periodic component in the spatial variability of r.

It is noted that, although the length of individual wetland and channel segments is variable, it is significantly greater than 1 km (Figure 5.8) and is therefore captured in the semi-variogram. A quasi-periodic behaviour for $\gamma_r(s)$ is consistent with the length of typical wetland and channel segments, the separation between measured cross-sections and the observation that the channel invert for the wetland category tends to be higher than that for the channel category.

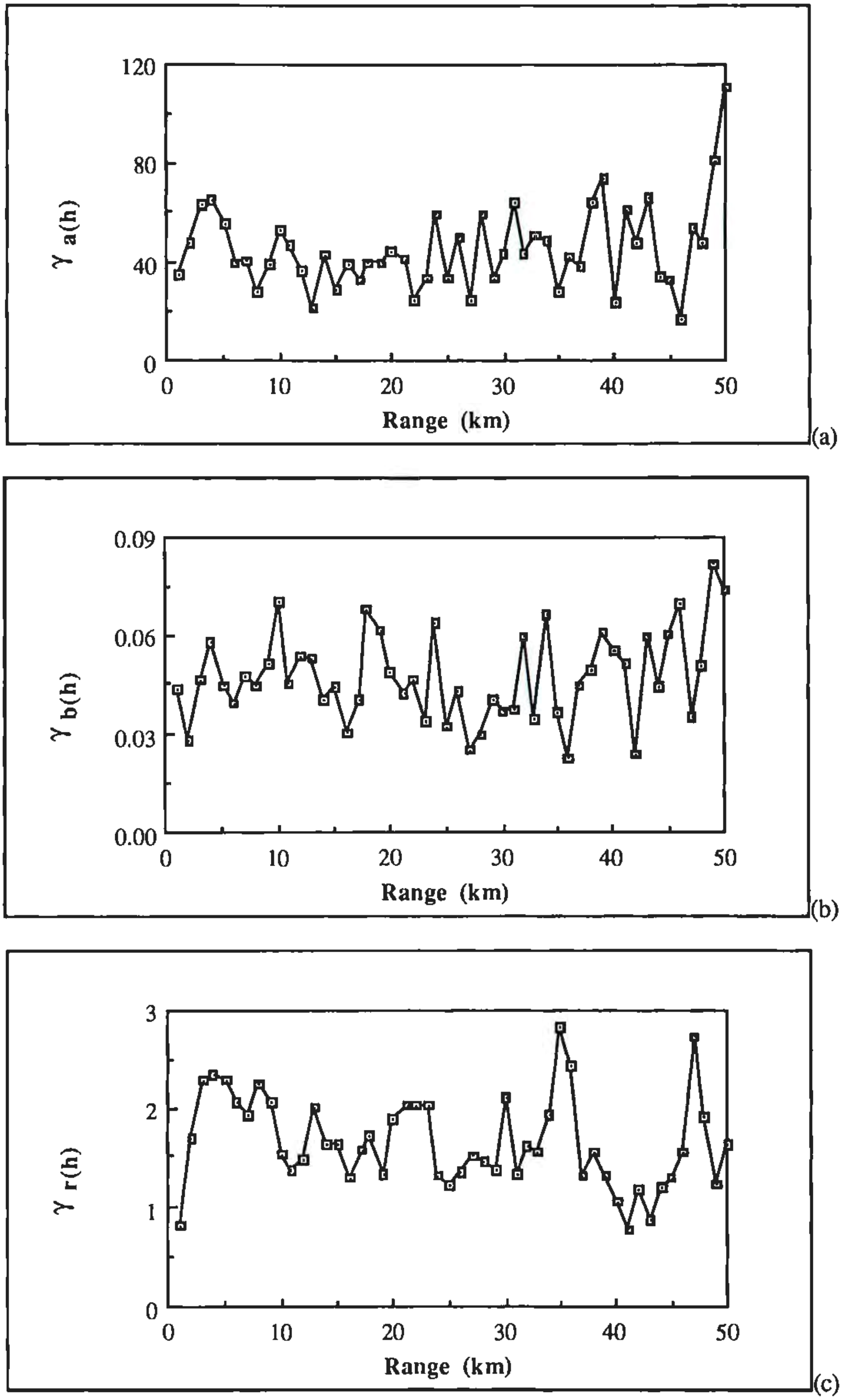


Figure 5.14: Semi-variograms showing the variability of cross-section parameters as a function of separation for Reach 2 and Reach 3.

5.3.3 CONCLUSIONS.

Cross-sections from the Wimmera River Channel can be described using four different parameters, a, b, r, and Z_{bf}, which are, a coefficient and index for a power curve relating width to elevation, a vertical displacement from the mean grade-line, and the bank-full depth respectively. The parameter a is considered log-normally distributed and the parameters b and r are considered normally distributed.

The Wimmera River channel can be statistically divided between areas in which an anabranching pattern is found and areas in which there is a single channel. There is also a statistically significant difference between upstream and downstream sections of the river.

Longitudinal trends in the river channel were examined. Moving downstream:

- the channel becomes shallower and the invert becomes more variable;

- the average slope decreases;

- the width to depth ratio increases; and

- the cross-sections become less V and more U shaped.

The internal structure of the data set was also examined. Significant cross-correlation exists between log(a) and b but not between log(a) and r or b and r. This correlation is generally maintained when the data are divided into three categories; however there is a tendency for greater cross-correlation in the upper section of the river. This is associated with a smaller channel variability and indicates that the variability in shape and vertical position of a cross-section is more systematic. Spatial variability of a, b and r was examined for the cross-sections at and downstream of Horsham.

5.4 A STOCHASTIC RIVER GEOMETRY MODEL.

Infilling cross-section data is often done by interpolating between measured cross-sections; however this approach neglects channel variability which is a significant feature of natural channels. A more rational way to infill cross-sectional data is to use a stochastic approach (Chiu and Lee, 1971). It is necessary for a stochastic model of a river channel to reflect the essential components of the channels variability; however this does not mean that such a model need be a perfectly refined input model. A simpler model, proven by sensitivity analysis to be adequate has the advantage of being easier to use (Chiu and Lee, 1971). A stochastic cross-section model incorporating along-channel variability was used to infill cross-sections for the MIKE 11 model of the Wimmera River.

This model is based on the findings in §5.3. Although the above analysis provides the basis of the model a number of additional assumptions are required and these are presented and justified in this section. These assumptions are made because the data set used in the above analysis does not cover the entire river. Therefore a method of extrapolating the statistical properties of the cross-sections to reaches of river without cross-sectional information was required. The model divides the river into two categories and applies appropriate distributions for the three cross-section parameters at each location.

The following assumptions were made:

1. r and b are normally distributed and a is log-normally distributed;

2. the conditional mean of b can be calculated from the value of a for a cross-section independently of channel type and location and the standard deviation of b about this conditional mean is constant;

3. there is zero serial correlation between cross-sections;

4. there is a linear trend in the mean of log(a) for each category while there are significant tributaries entering the river (ie. upstream of Norton's Creek) and the mean of a for each category is constant downstream of the last significant tributary;

5. log(a) has a constant coefficient of variation for each category;

6. the standard deviation of r can be treated analogously to the mean of log(a);

7. the mean of r is a constant proportion of the standard deviation of r for each category;

8. the ratios of the mean of log(a) to the standard deviation of r for wetlands and channels do not vary along the river.

	Wetland	Channel
$\mu_{log(a)}$	0.57 + 0.0039 Ch Ch < 116Km 1.02 Ch >116Km	0.67 + 0.0045 Ch Ch <116Km 1.20 Ch>116Km
$\sigma_{log(a)}$	0.14 $\mu_{log(a)}$	0.14 $\mu_{log(a)}$
μ_b	1.37 - 0.63 log(a)	1.37 - 0.63 log(a)
σ_b	0.18	0.18
μ_r	0.86 σ_r	-0.34 σ_r
σ_r	0.028 + 0.0092 Ch Ch <116Km 1.19 Ch>116Km	0.030 + 0.010 Ch Ch<116Km 1.09 Ch>116Km

Table 5.4: Calculation of mean and standard deviation of cross-section parameters used to generate stochastic cross-sections.

Assumption 7 is the same as r having a constant coefficient of variation; however, by using the inverse of the coefficient of variation, the problem of dealing with a zero mean or infinite coefficient of variation can be avoided. Assumption 6 deals with σ_r rather than μ_r for the same reason.

Table 5.4 contains the equations used to calculate the mean and standard deviation of log(a), b and r. Assumptions 4 - 8 are made so that the properties of the cross-section along the river may be simply interpolated and so that those properties capture the main features of the river channel. Figure 5.6 and Table 5.3 provide evidence to qualitatively support these assumptions. Assumptions 1 - 3 relate to the distributions of the cross-section parameters and are justified in §5.3.2.

A computer program was written to generate cross-sections at the desired locations based on the above assumptions and the equations in Table 5.4. The method of generation used in the program is as follows:

1. the location at which the cross-section is required is determined and from this the channel category at that location is determined;
2. using this information the mean of log(a) and the standard deviation of r are calculated;
3. using assumptions 5 and 7 the standard deviation of log(a) and the mean of r are calculated;
4. a value of log(a) is generated using a normal distribution;

5. using assumption 2 the mean of b is calculated;

6. using the calculated parameters and a normal distribution values of b and r are generated;

7. the mean channel invert and top of bank are determined from the chainage;

8. log(a) is converted to a and the channel invert is calculated from r and the mean channel invert;

9. hydraulic properties are then calculated at elevations determined using an exponential scale between the invert and top of bank and printed in tabular form compatible with MIKE 11.

5.4.1 QUALITY OF STOCHASTIC GENERATION.

Due to the small size of the data set used to develop the stochastic model of the Wimmera River Channel it is not possible to perform a split sample test such as recommended by Klemes (1986) to evaluate the performance of the model. It is possible to compare the generated cross-sections to the cross-sections used to develop the model; however such a test is so weak that it is not considered useful. The model was developed using a rigorous statistical analysis of the available data and cannot be directly tested due to the small sample size. This is a short coming that cannot be overcome without measuring additional cross-sections along the river. However the importance of including channel variability in a hydrodynamic model of a stream like the Wimmera River can be assessed using numerical experimentation techniques.

5.5 HYDRAULIC IMPACT OF CHANNEL VARIABILITY.

The impact of along channel variability in cross-section shape and invert level was examined by generating a series of stochastic river channels which were used as the bases for eight hydraulic models. These river channels differed only in the variability of the three parameters, a, b and r, used to define individual cross-sections.

Once the hydraulic models had been established numerical simulations using steady and unsteady discharges were conducted. The steady flow simulations were for discharges of 1 m^3/s, 10 m^3/s and 100 m^3/s and the unsteady discharge consisted of an artificial hydrograph. Following the numerical simulations the results from each of the eight models were compared for each flow case. This allowed the effects of σ_a, σ_b and σ_r to be quantified. A factorial framework (Chatfield, 1983), which has the advantage of allowing interactions between variables to be assessed, was used to analyse the results.

5.5.1 METHODOLOGY.

5.5.1.1 The Complete Factorial Approach.

The complete factorial method used requires that each variable or factor have two levels, nominally low and high. This approach is useful for screening experiments which aim to identify important variables. Experiments are conducted for all combinations of variables. Results from the complete factorial experiment can be analysed to estimate the impact of individual variables, which are called main effects, and to estimate the interactions between variables (Chatfield, 1983).

In this case the standard deviations of each of the cross-section parameters, σ_a, σ_b and σ_r, were treated as variables representing the along channel variability of the channel cross-section. The different channel geometries are referred to as follows. Three letters are used corresponding to σ_a, σ_b and σ_r respectively and high variability cases are represented with h and low variability with l. Thus lhl refers to a channel with low σ_a, high σ_b and low σ_r. The main effects of σ_a, σ_b and σ_r are denoted with A, B and R and interactions between σ_a and σ_b, σ_a and σ_r, σ_b and σ_r and σ_a, σ_b and σ_r, which are referred to as AB, AR, BR and ABR, following Chatfield (1983).

For an experiment with three factors, each having two levels, the main effects and interactions can be calculated using Table 5.5. To calculate the effect or interaction, the appropriate line is selected from Table 5.5 and the results from each simulation are added or subtracted as indicated. The result is then divided by 4 since each effect or interaction is calculated on the basis of four pairs of experiments (Chatfield, 1983).

5.5.1.2 Stochastic Channel Generation.

The synthetic river channels used in the numerical simulations were generated as follows. A mean grade of 1:2000 was assumed which is similar to the mean slope of the Wimmera River. A 110 km river reach was used for which cross-sections

Effect	lll	hll	lhl	hhl	llh	hlh	lhh	hhh
A	-	+	-	+	-	+	-	+
B	-	-	+	+	-	-	+	+
R	-	-	-	-	+	+	+	+
AB	+	-	-	+	+	-	-	+
AR	+	-	+	-	-	+	-	+
BR	+	+	-	-	-	-	+	+
ABR	-	+	+	-	+	-	-	+

Table 5.5: Calculation of main effects and interactions.

were generated every 1 km. The results from the lowest 10 km of river channel were discarded so that the downstream boundary condition would not affect the analysis. An interpolated cross-section was added between each generated cross-section by MIKE 11. Eight channels were generated with all the possible combinations of low and high variability for a, b and r as discussed in the previous section. High values of variability were chosen to be representative of cross-sections from the Wimmera River and low values of variability were simply 50% of the high variability case. The means of a and b were chosen to be representative of cross-sections from the Wimmera River. The means, μ_a and μ_b, and standard deviations, σ_a, σ_b and σ_r, used for each channel are given in Tables 5.6 and 5.7. The mean of r was zero. No attempt was made to include the effect of the variation between channel and wetland areas in this experiment.

A normal distribution was used to generate r. The distribution of a and b is a bivariate lognormal-normal distribution and the following methodology was used to generate them. A value of log(a) was generated from a normal distribution using the high value of the standard deviation ie. the value typical for the Wimmera River. The mean of b was then calculated using:

$$\mu_b = 1.37 - 0.63 * \log (a) . \qquad (5.11)$$

A value of b was then generated from that mean and the appropriate standard deviation for the particular case and the value of log(a) was adjusted if the channel was being generated with a low variability of a. It should be noted that this adjustment was made so that the σ_a was halved and μ_a was preserved in the real domain. This method of generation means that the variability of b used was not affected by changes in the variability of a and vice versa. The same set of standard normal (N(0,1)) random variates was used to generate all eight channels so that the

		Standard Deviation		Coefficient of Variation	
Parameter	Mean	Low	High	Low	High
a	14.4	1.92	3.83	0.117	0.235
b	0.61	0.146	0.233	0.237	0.379

Table 5.6: Mean, standard deviation and coefficient of variation for cross-section parameters a and b.

		Standard Deviation		$\sigma_r / \bar{y}$	
Q (m^3/s)	$\bar{y}$	Low	High	Low	High
1	1.04	0.545	1.090	0.52	1.05
10	2.23	0.545	1.090	0.24	0.49
100	5.44	0.545	1.090	0.10	0.20

Table 5.7: Mean depth, standard deviation and variability of channel invert for steady flows of 1 m^3/s, 10 m^3/s and 100 m^3/s.

only difference between the channels was related to the change in variance.

5.5.1.3 Application of MIKE 11.

A simple single channel numerical model was established using the MIKE 11 package. An inflow at the upstream end of the artificial river reach and an arbitrary stage-discharge relationship at the downstream end were used as boundary conditions. Solutions were calculated using the complete St Venant Equations and standard numerical parameters for the solution scheme incorporated in MIKE 11. Flow resistance was simulated using Manning's relationship and a value of 0.06 for n. No allowance is made for local losses in MIKE 11. A 2 minute time-step was required to ensure stability of the simulation. For steady state solutions the model was run until flow and stage were constant at each computational point. The initial conditions for the unsteady flow simulations were simply the steady state solutions for the appropriate flow rate.

5.5.2 STEADY FLOW SIMULATIONS.

The first series of simulations conducted were for steady flows of 1 m^3/s, 10 m^3/s and 100 m^3/s. These flow rates represent relatively low, medium and high (approximately bank-full) discharge in the Wimmera River and thus in the generated channels. Simulations for each of these flow rates are treated as separate experiments.

The choice of variables to represent the hydraulic response is now discussed and the results analysed. The analysed is conducted in two stages. Firstly the mean (averaged over 100 cross-sections) hydraulic response to changes in along channel variability is examined then the response at individual cross-sections is examined.

5.5.2.1 Representation of Hydraulic Response.

Three variables were chosen to represent the hydraulic response at each cross-section: the depth, stage and residence time. Depth is simply the maximum water depth at the cross-section. Stage is the elevation of the water above the cease to flow elevation which is the highest invert elevation at or downstream of the location where the water level is predicted. Water levels predicted by MIKE 11 at each cross-section were reduced to depth and stage at each cross-section. Residence times were calculated for the 1 km reach represented by each cross-section by dividing the reach length by the cross-sectional mean velocity predicted by MIKE 11.

These three variables were chosen to represent the hydraulic response because they represent different aspects of that response. Firstly, the depth of water at a specific cross-section is of interest in its own right and is of relevance when considering, for example, fish habitat. Secondly, stage is analogous to the stage often used in river gauging and variations in stage with discharge are important when considering transient storage which has a major influence on flow routing. Finally, from a solute transport perspective, residence time is important.

5.5.2.2 Mean Response.

The mean effects and interactions were calculated separately for each discharge as follows:

1. calculate depth, stage and residence time at each cross-section in the eight stochastic river channels;

2. calculate the (channel) mean depth, stage and residence time for each river channel;

3. calculate the main effects and interactions from the channel mean depths, stages and residence times.

4. calculate the global mean depth, stage and residence time using the eight channel means.

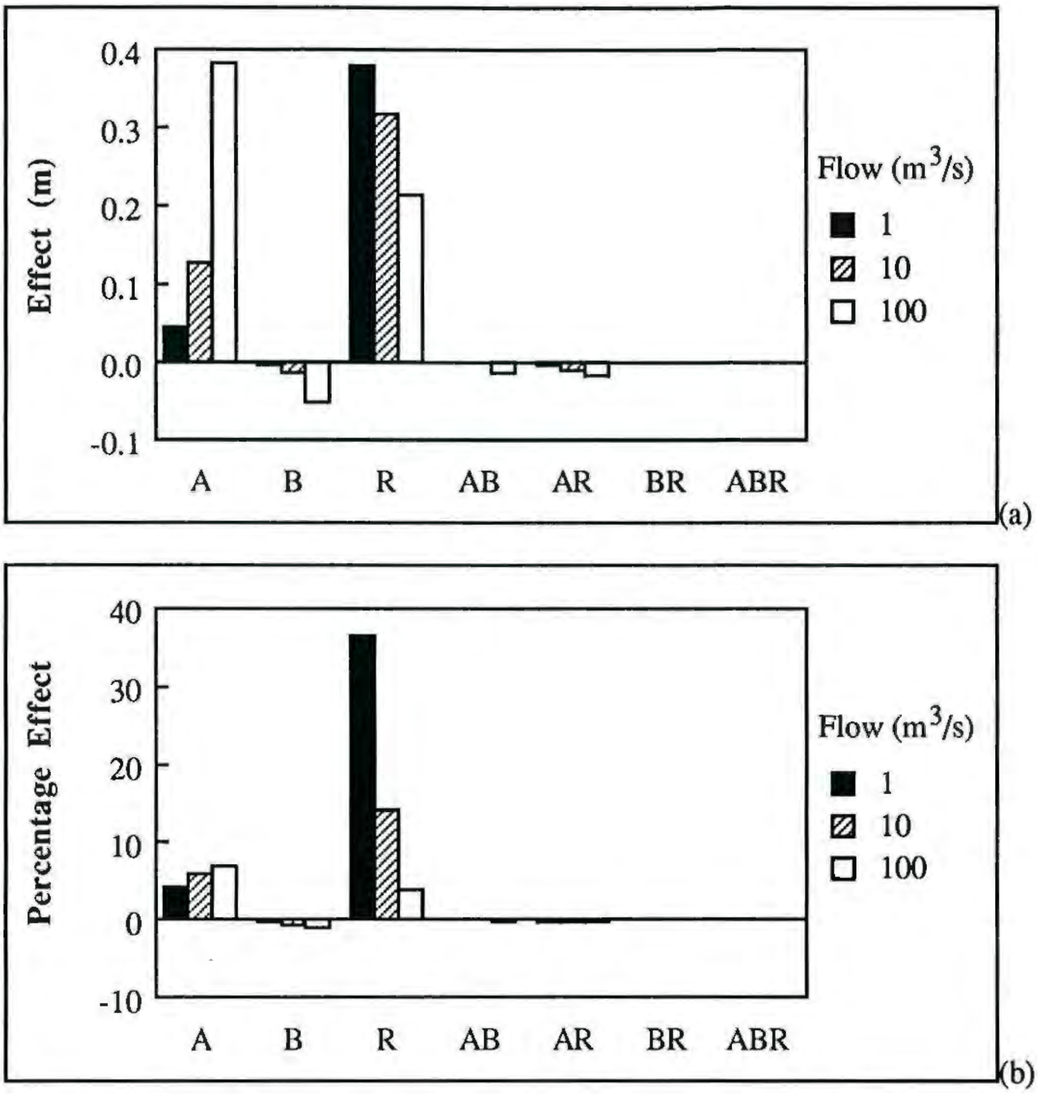


Figure 5.15: (a) The average effect of channel variability on depth as a function of discharge. (b) The average effect of channel variability on depth expressed as a percentage of average depth as a function of discharge.

5. normalise the interactions and main effects using the appropriate global mean.

Figure 5.15, 5.16 and 5.17 show the main effects and interactions at the three different flow rates for depth stage and residence time. Main effects and interactions are also shown as percentages of the global mean depth, stage and residence time. Interactions between σ_a, σ_b and σ_r are unimportant in determining the mean hydraulic behaviour of the channel. Given that B is small compared to A and R it can also be concluded that variability of channel shape is relatively unimportant in determining the hydraulic behaviour.

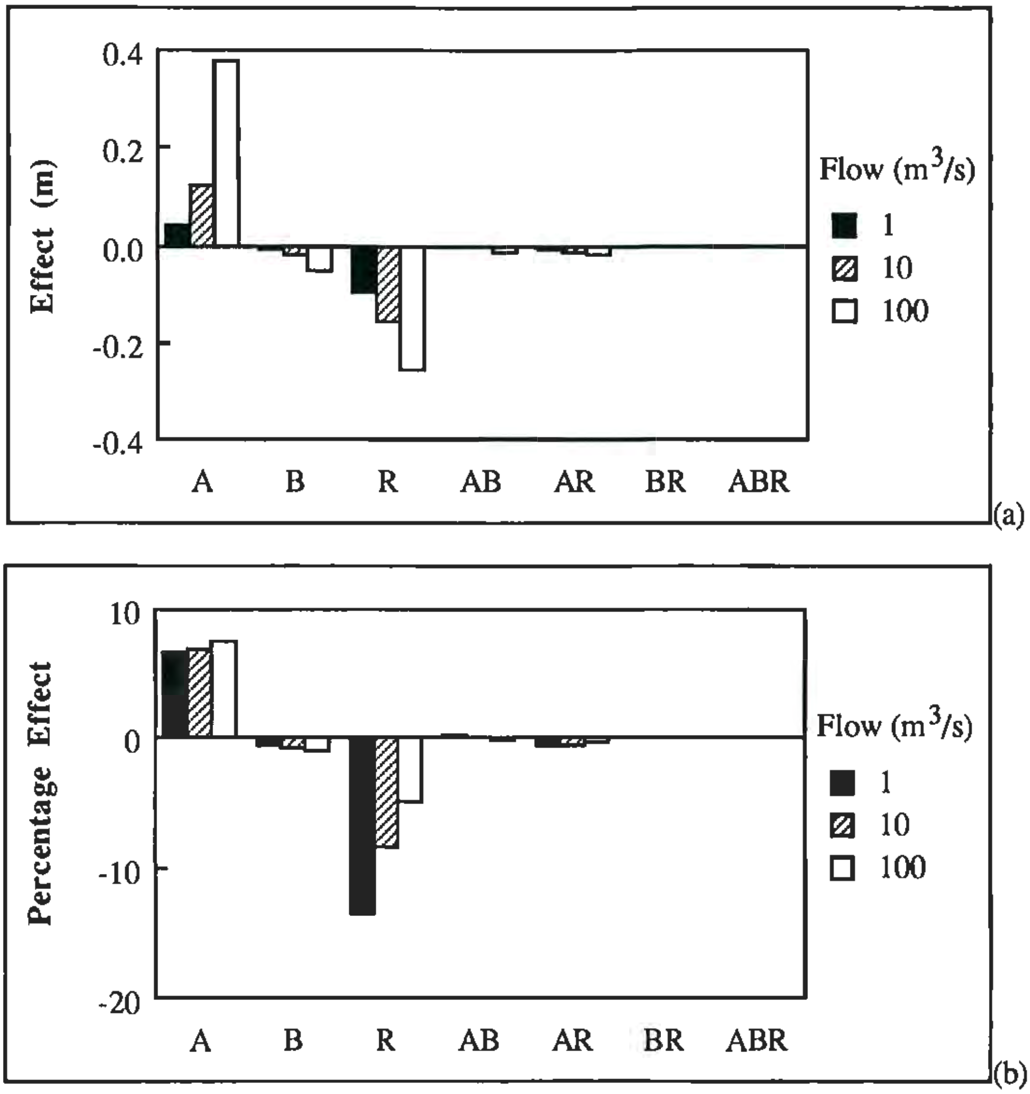


Figure 5.16: (a) The average effect of channel variability on stage as a function of discharge. (b) The average effect of channel variability on stage expressed as a percentage of average stage as a function of discharge.

Figures 5.15 and 5.16 indicate that σ_a and σ_r have the greatest influence on depth and stage. The processes by which σ_a and σ_r influence the hydraulic response are discussed in §5.5.2.4. The largest effect for mean stage or depth was R for low flows. It should be noted that the change in σ_a and σ_r was 67% compared to the mean standard deviation and that A and R were generally small in comparison.

5.5.2.3 Local Response.

The local changes in hydraulic response due to changes in channel variability were also considered. The mean effects and interactions at specific cross-sections were calculated for each discharge as follows:

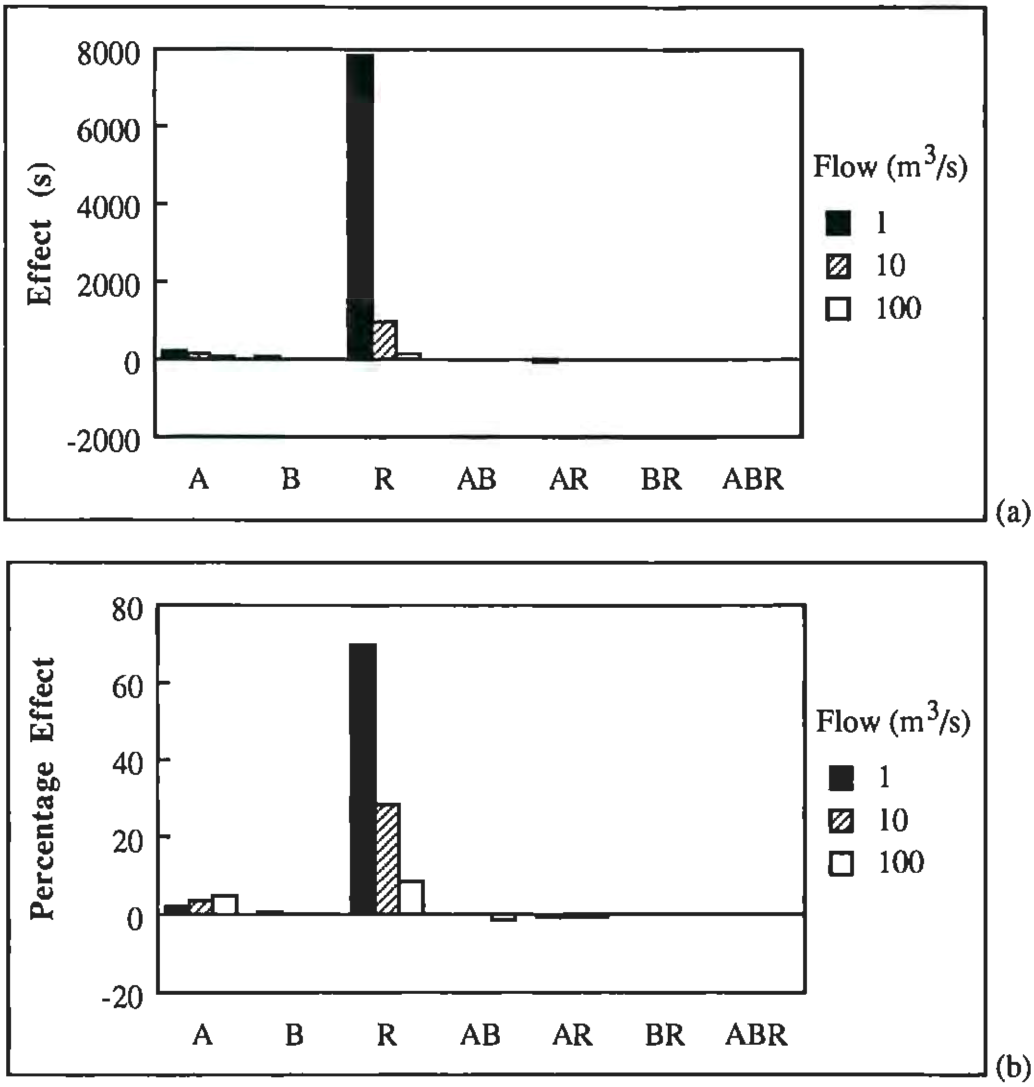


Figure 5.17: (a) The average effect of channel variability on residence time as a function of discharge. (b) The average effect of channel variability on residence time expressed as a percentage of average residence time as a function of discharge.

1. calculate depth, stage and residence time at the corresponding cross-section in each of the eight stochastic river channels;

2. calculate the main effects and interactions from the depths, stages and residence times.

3. calculate the mean depth, stage and residence time at that cross-section;

4. normalise the interactions and main effects for each cross-section using the appropriate cross-sectional mean.

This provided 100 realisations of the main effects and interactions for each flow rate and each variable. Table 5.8 provides the mean, standard deviation and the minimum and maximum of the normalised effects and interactions for each flow rate and response variable. It should be noted that the statistics provided in Table 5.8 were calculated from the normalised main effects and interactions at each cross-section. The normalised main effects (A, B and R) represent changes in the depth, stage or residence time at a cross-section that are attributable to changes in σ_a, σ_b and σ_r as a percentage of the local mean depth, stage or residence time as appropriate. The normalised interactions (AB, AR, BR, ABR) represent changes in the hydraulic response at a cross-section attributable to interactions between σ_a, σ_b and σ_r as a percentage of the local mean response. When examining Table 5.8, it should be remembered that the changes in σ_a, σ_b and σ_r were 67% of the average standard deviation.

An examination of Table 5.8 indicates that the magnitude of the mean and standard deviation of the interactions is generally smaller than those for the main effects and the magnitudes of mean and standard deviation of A and R are generally larger than those for B. This indicates that the interactions are less important than the main effects and that, of the main effects, A and R are generally more important than B at individual cross-sections. Furthermore the mean values for A and R are generally small compared to the changes in the channel variability, which indicates that the hydraulic response, when averaged along the channel, is insensitive to the channel variability. It is noted that similar results were obtained when the mean effects for the whole channel were examined (§5.5.2.2).

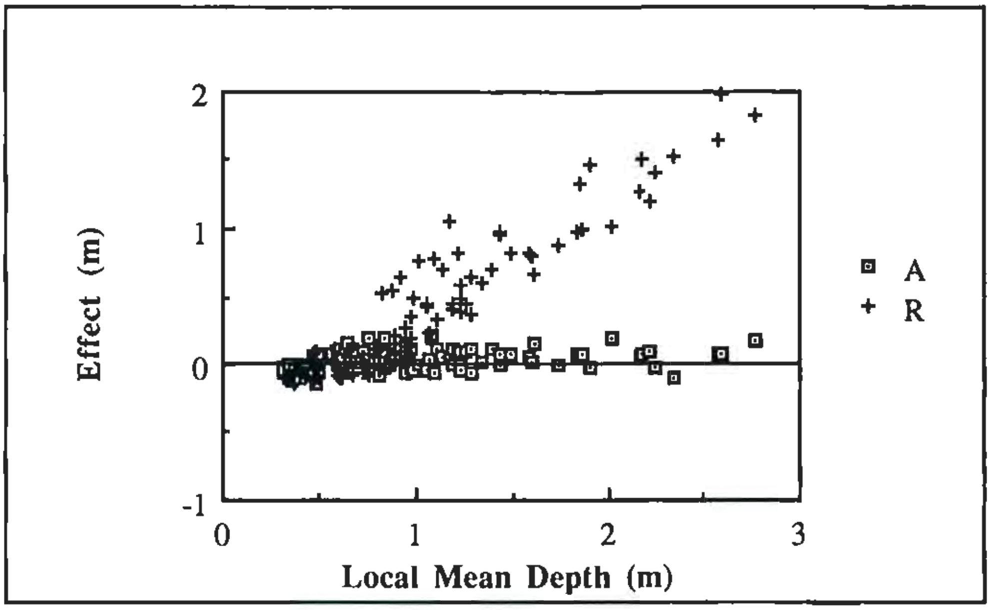
While the average hydraulic response may be insensitive to channel variability, Table 5.7 indicates that A and R are highly variable (coefficient of variation is almost always greater than 1). The minimum and maximum values of A and R indicate that the response at specific cross-sections can be sensitive to changes in channel variability. The behaviour of A and R is now considered in greater detail.

5.5.2.4 Hydraulic Processes and Channel Variability.

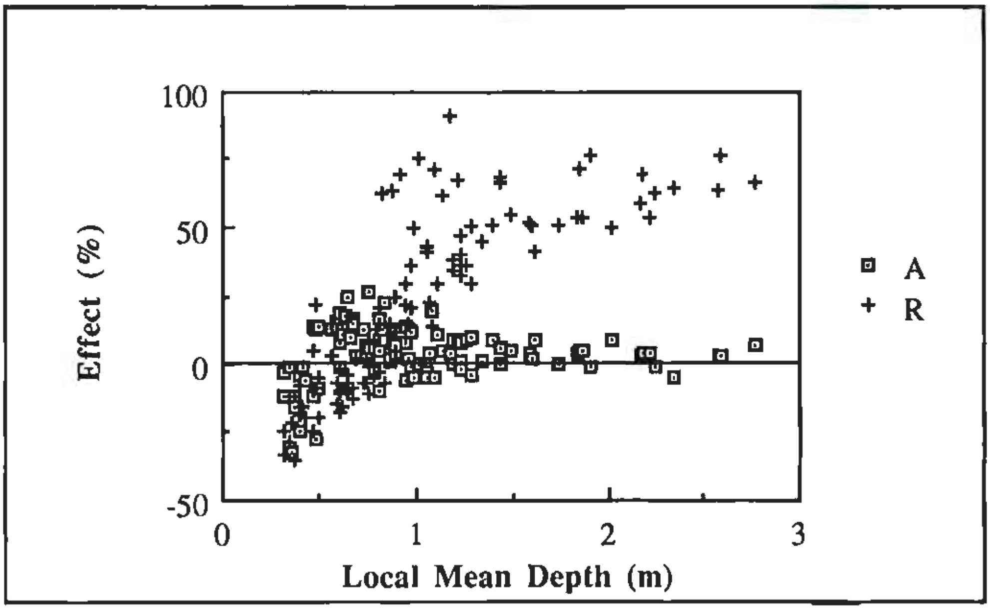
The change in depth (as indicated by A and R) due to changes in σ_a and σ_r is considered first. Figures 5.18, 5.19 and 5.20 show A and R as a function of the mean depth at each cross-section. For a given flow R is negative at cross-sections where the flow is shallow. At deeper cross-sections R becomes positive and increases. R tends to become smaller (more negative) in absolute terms at all cross-sections at higher flows. The normalised value of R is approximately

		Main Effects			Interactions			
		A	B	R	AB	AR	BR	ABR
Depth $1\ m^3/s$	μ (%)	3.6	-1.2	22.6	0.2	-0.3	0.1	0.1
	σ (%)	10.6	6.2	32.5	0.5	2.3	1.0	0.3
	min (%)	-32.1	-34.5	-35.5	-1.5	-10.5	-3.4	-0.7
	max (%)	26.4	8.2	91.0	2.8	9.6	3.4	1.4
Depth $10\ m^3/s$	μ (%)	5.5	-0.8	9.7	0.0	-0.5	0.0	0.0
	σ (%)	8.2	1.6	17.2	0.2	1.4	0.4	0.1
	min (%)	-14.7	-8.9	-23.5	-0.6	-5.0	-1.1	-0.3
	max (%)	26.2	0.9	47.0	0.8	3.9	1.0	0.3
Depth $100\ m^3/s$	μ (%)	6.9	-1.0	2.8	-0.2	-0.3	0.0	0.0
	σ (%)	6.4	1.5	8.8	0.3	0.7	0.1	0.0
	min (%)	-6.1	-4.3	-18.4	-1.1	-3.3	-0.2	-0.1
	max (%)	23.0	3.6	20.8	1.0	1.5	0.4	0.1
Stage $1\ m^3/s$	μ (%)	4.9	-1.6	-14.9	0.2	-0.5	0.1	0.1
	σ (%)	12.1	6.9	16.8	0.6	3.4	1.5	0.4
	min (%)	-32.1	-34.5	-72.6	-1.5	-17.3	-6.9	-0.7
	max (%)	26.4	8.2	21.7	2.8	9.6	4.9	1.4
Stage $10\ m^3/s$	μ (%)	6.2	-0.9	-9.0	0.0	-0.6	0.0	0.0
	σ (%)	8.9	1.8	9.8	0.2	1.6	0.4	0.1
	min (%)	-14.7	-8.9	-33.7	-0.6	-6.1	-1.5	-0.3
	max (%)	26.2	0.9	13.9	0.8	3.9	1.0	0.3
Stage $100\ m^3/s$	μ (%)	7.3	-1.0	-5.4	-0.3	-0.4	0.0	0.0
	σ (%)	6.6	1.6	5.8	0.4	0.7	0.1	0.0
	min (%)	-7.3	-4.3	-18.4	-1.1	-3.3	-0.3	-0.1
	max (%)	23.0	3.6	6.7	1.0	1.5	0.4	0.1
Residence time $1\ m^3/s$	μ (%)	-4.8	-0.2	36.1	0.3	-2.4	-0.3	0.0
	σ (%)	25.4	4.0	48.1	0.9	11.2	1.9	0.7
	min (%)	-66.4	-13.6	-38.6	-1.9	-41.8	-5.9	-2.2
	max (%)	53.3	9.4	124.6	4.3	33.1	6.1	3.0
Residence time $10\ m^3/s$	μ (%)	-2.2	0.1	16.0	-0.1	-1.5	0.0	-0.1
	σ (%)	27.3	3.8	27.6	0.6	6.0	1.5	0.3
	min (%)	-58.1	-9.9	-35.4	-2.4	-20.8	-5.0	-1.9
	max (%)	67.1	12.4	80.4	1.7	14.9	5.5	0.6
Residence time $100\ m^3/s$	μ (%)	0.1	-0.1	4.8	-1.3	-0.7	0.1	-0.1
	σ (%)	29.9	11.9	14.7	1.8	3.0	1.3	0.3
	min (%)	-65.8	-23.1	-32.0	-8.9	-9.3	-2.8	-1.2
	max (%)	76.7	30.3	37.3	3.2	7.8	4.8	0.6

Table 5.8: Characteristics of the distribution of local effects of channel variability on depth, stage and residence time for different flows.



(a)

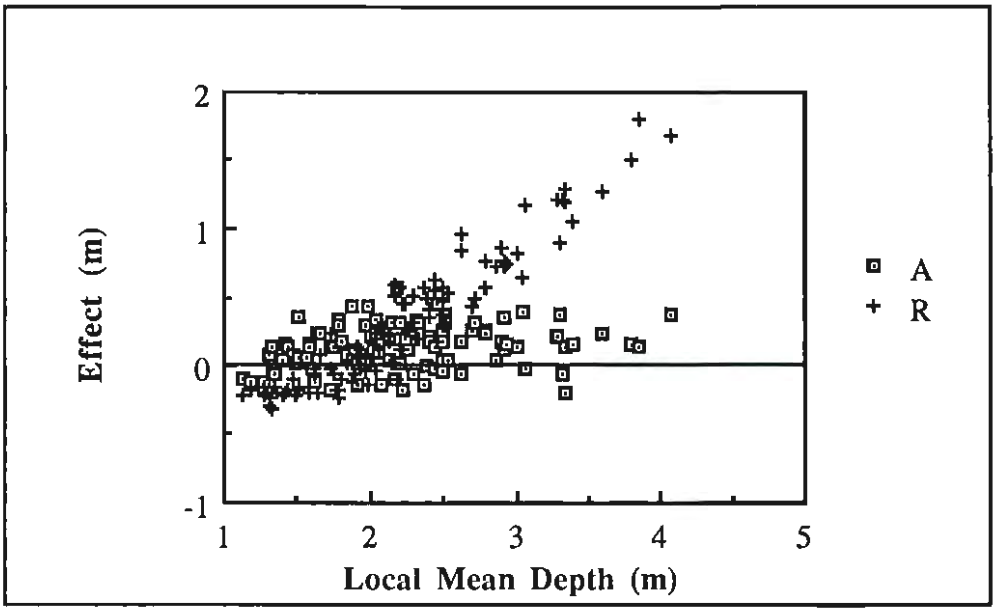


(b)

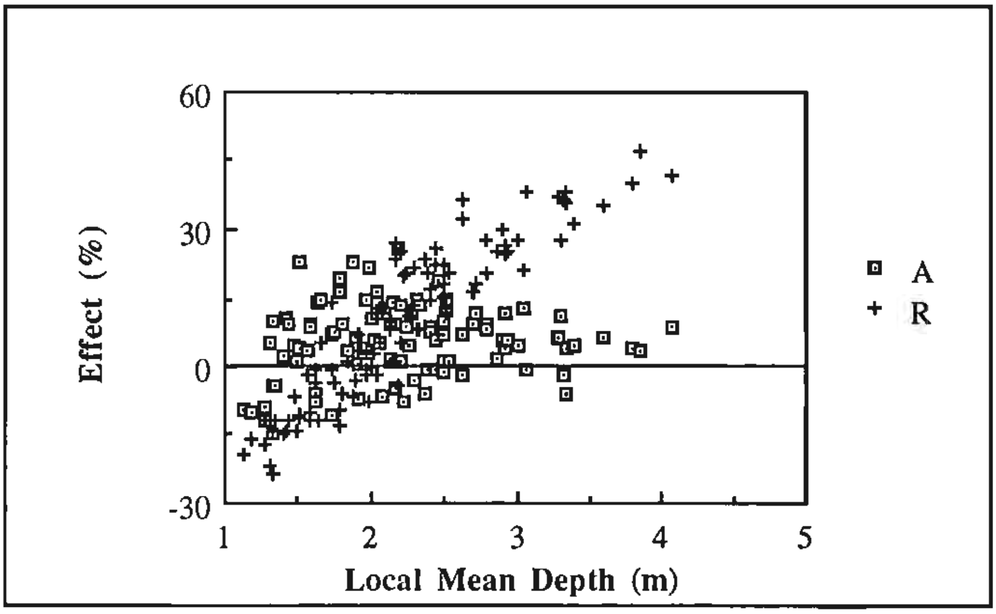
Figure 5.18: The effect of σ_a and σ_r on depth at individual cross-sections as a function of the mean depth at the cross-section for a discharge of 1 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean depth at that cross-section.

constant for all three flows for the shallow cross-sections but becomes smaller as the flow increases at deeper cross-sections.

This behaviour can be explained as follows. In shallow sections, backwater effects are not important and the depth is controlled locally. As the channel variability increases so does the bed slope in these sections, hence the depth will actually reduce and R will be negative. However in areas where backwater effects are important the depth is strongly influenced by a downstream constriction(s)



(a)



(b)

Figure 5.19: The effect of σ_a and σ_r on depth at individual cross-sections as a function of the mean depth at the cross-section for a discharge of 10 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean depth at that cross-section.

(shallow or narrow cross-section). If this downstream constriction is a high point in the bed (shallow cross-section) then it is the height of channel bed at the downstream section relative to the local section which is dominant in determining the depth and hence R. When the flow rate is small enough and the local depth large enough, R asymptotes to a constant proportion of the mean local depth

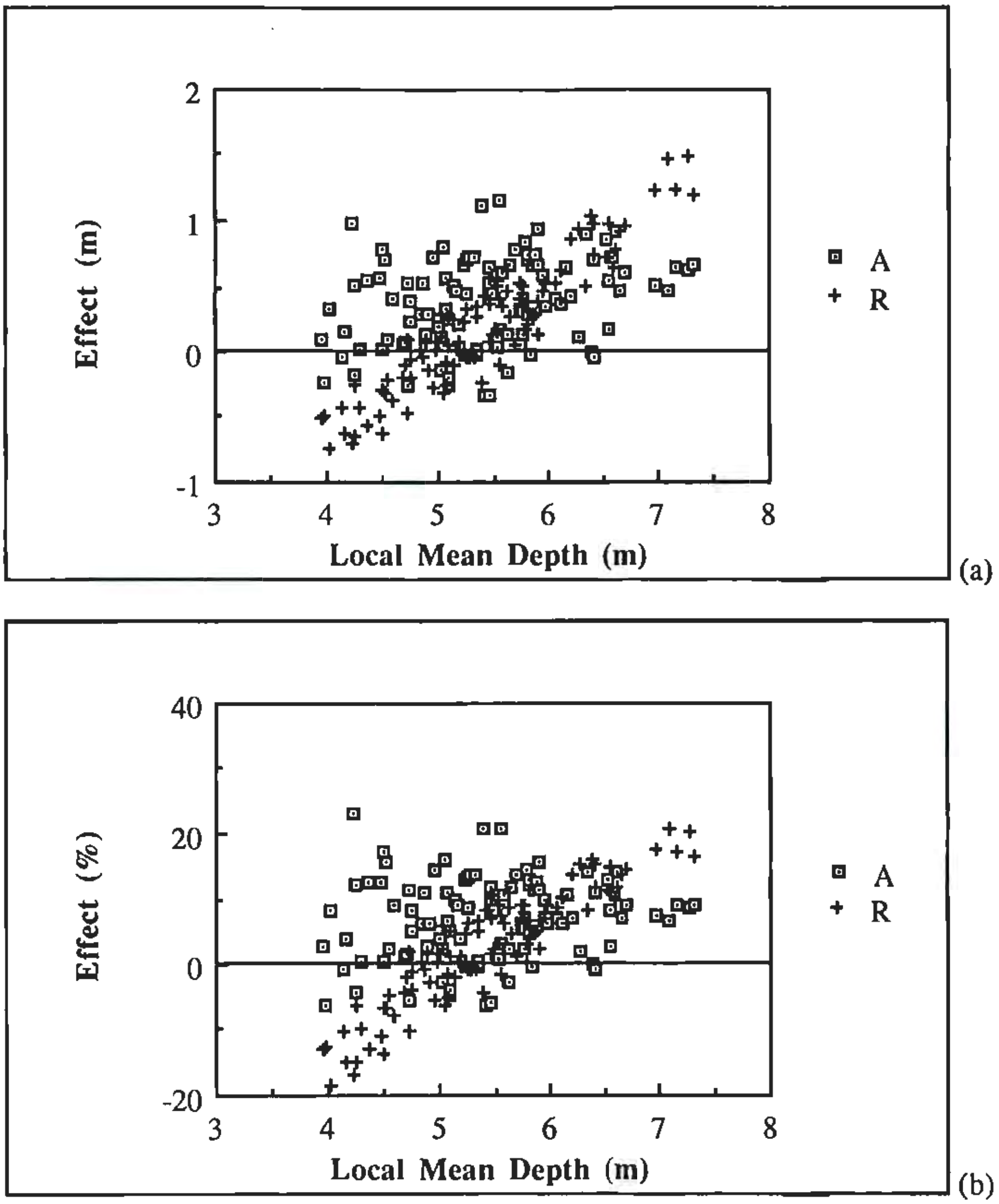
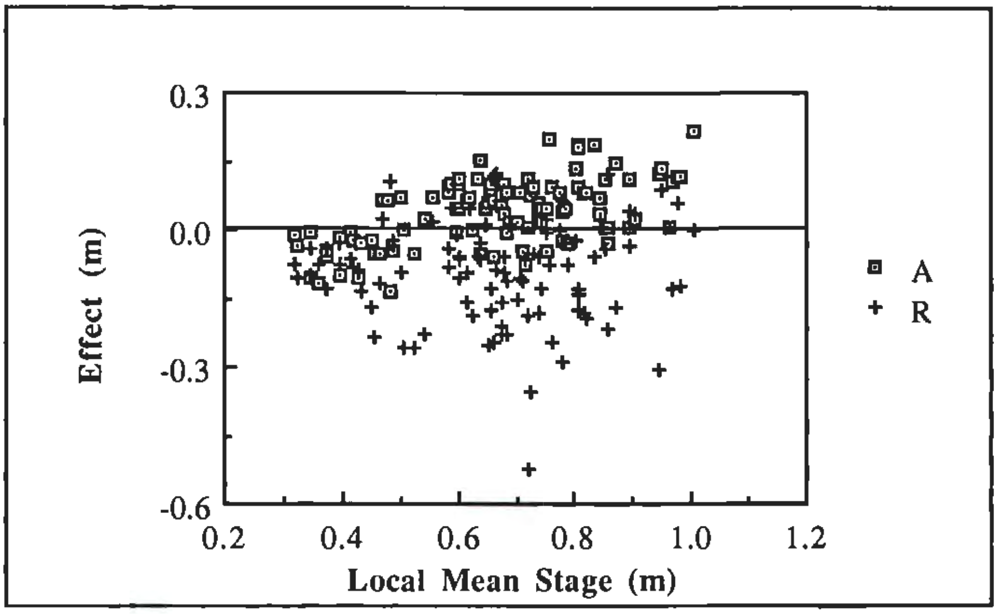


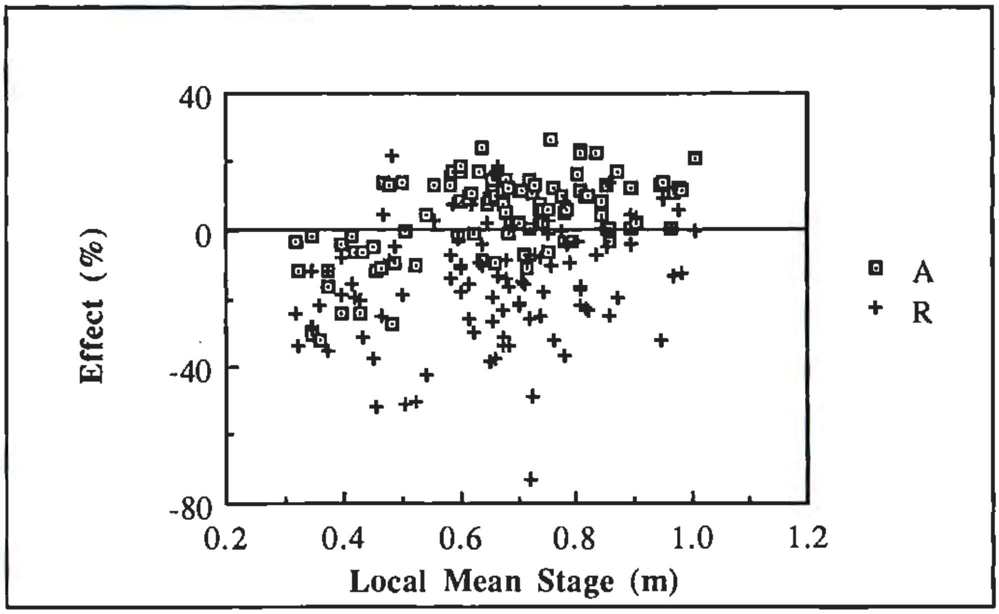
Figure 5.20: The effect of σ_a and σ_r on depth at individual cross-sections as a function of the mean depth at the cross-section for a discharge of 100 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean depth at that cross-section.

(Figure 5.18b). This indicates that the water surface profile is dominated by controls at topographic high points.

In contrast to R, A is not related to the local depth and it increases as the flow rate increases (Figures 5.18, 5.19, 5.20). Narrow cross-sections (small width to depth ratio) tend to cause backwater effects because the cross-sectional area is small.



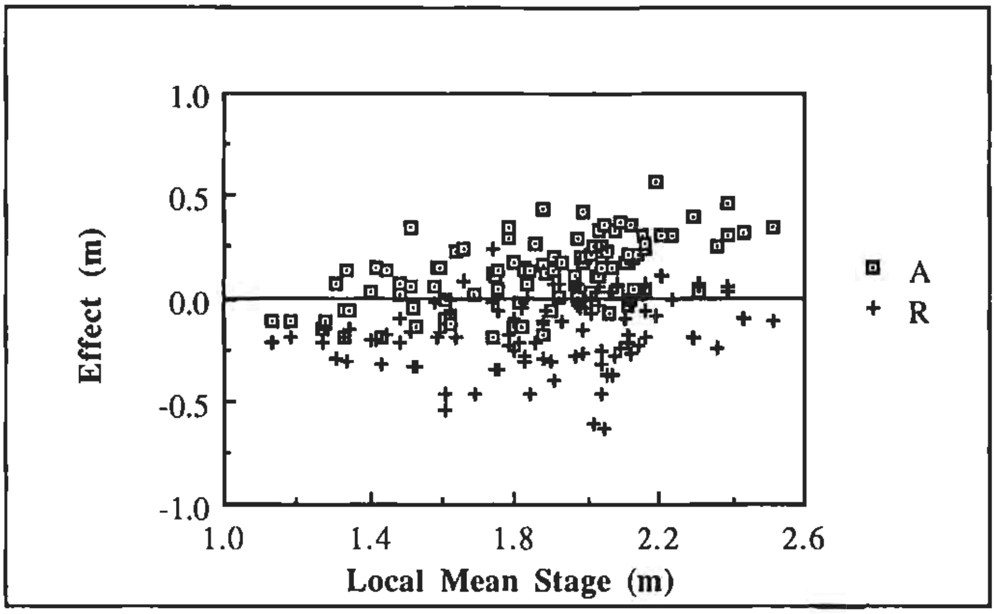
(a)



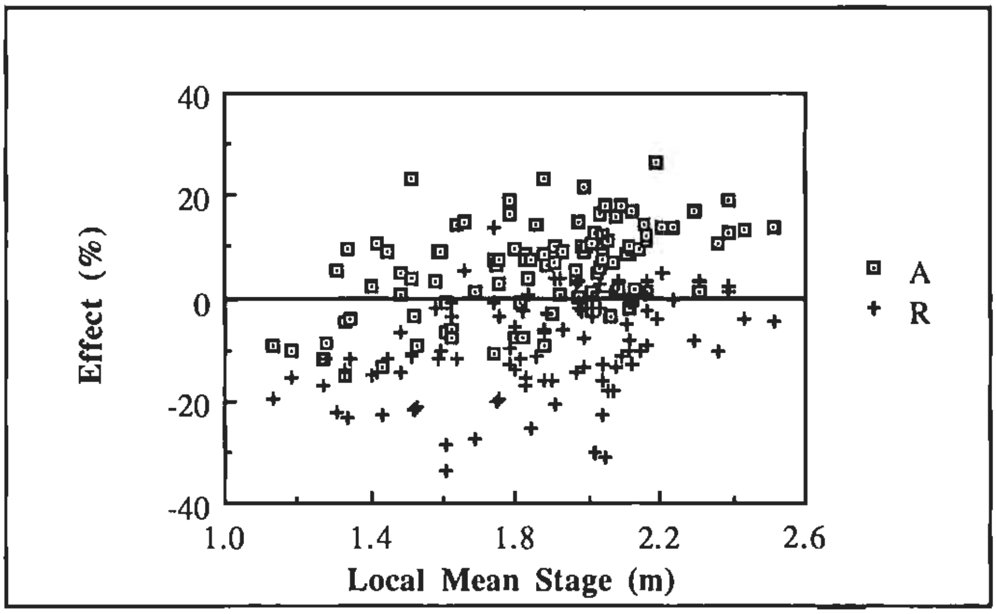
(b)

Figure 5.21: The effect of σ_a and σ_r on stage at individual cross-sections as a function of the mean stage at the cross-section for a discharge of 1 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean stage at that cross-section.

These cross-sections are associated with small values of a and tend to become narrower when σ_a is increased. Hence A is positive. However there is no correlation between a and r in the cross-section model used (nor in the Wimmera River), so narrow cross-sections are not necessarily associated with shallow flow. Therefore there is not a correlation between A and the depth of flow.



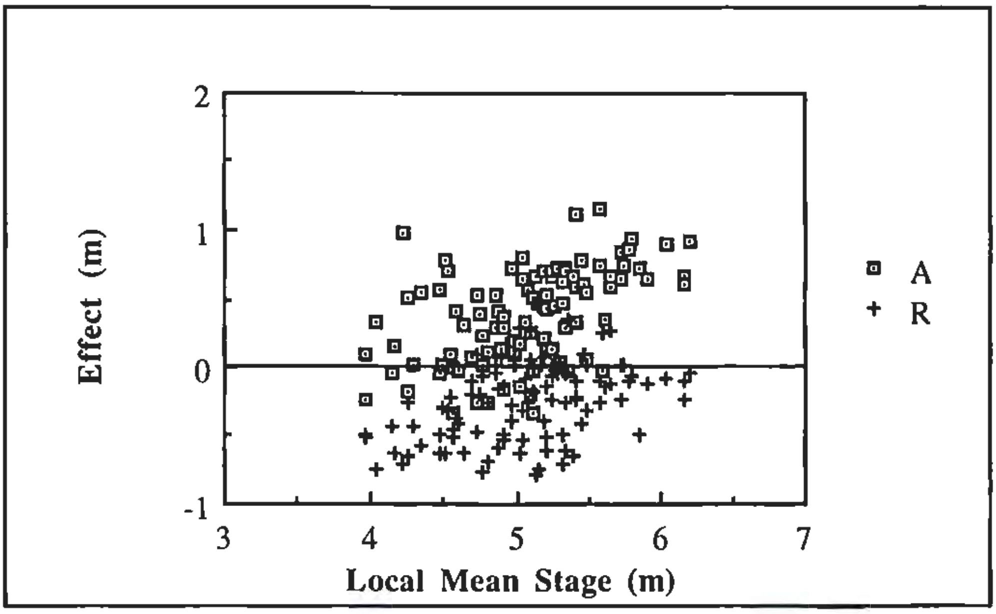
(a)



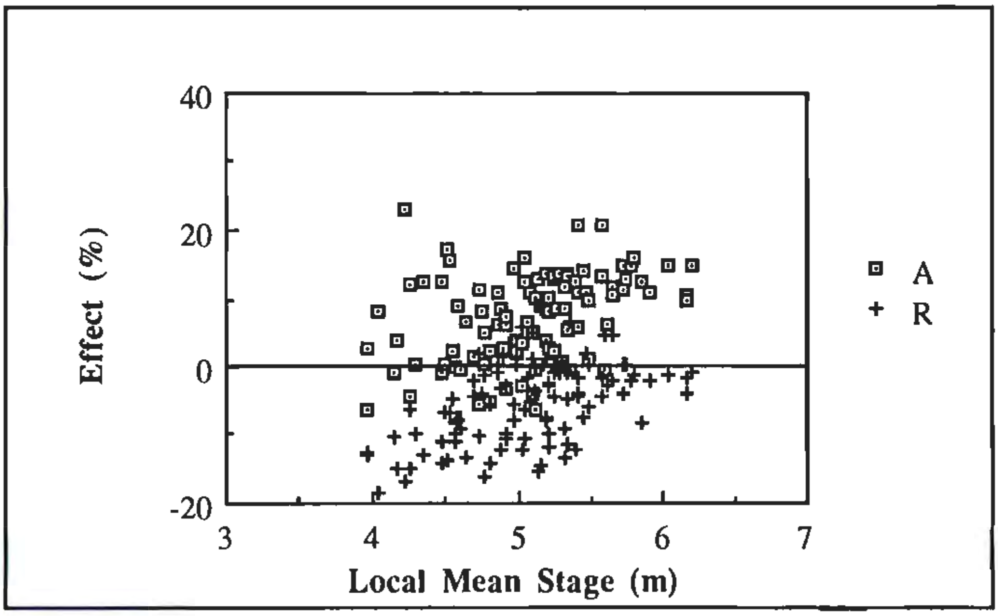
(b)

Figure 5.22: The effect of σ_a and σ_r on stage at individual cross-sections as a function of the mean stage at the cross-section for a discharge of 10 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean stage at that cross-section.

Figure 5.15a shows that, for depth, A becomes larger (in absolute terms) and R becomes smaller as the flow increases. This implies that narrow cross-sections (small width to depth ratio) become more important in controlling the depth and that high points in the river bed become less important. At low flows, high points are dominant because they are invariably associated with a small cross-sectional



(a)

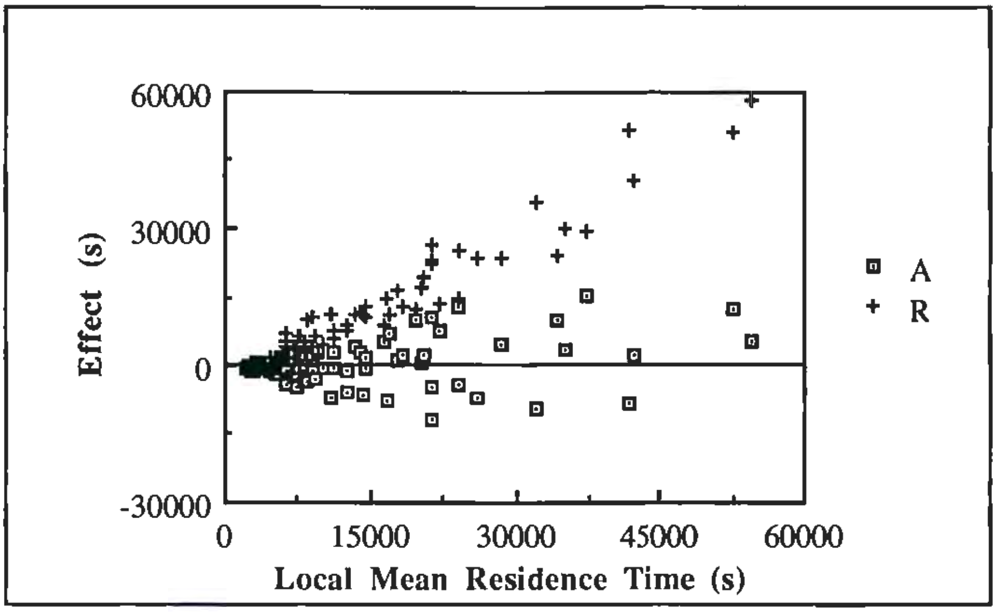


(b)

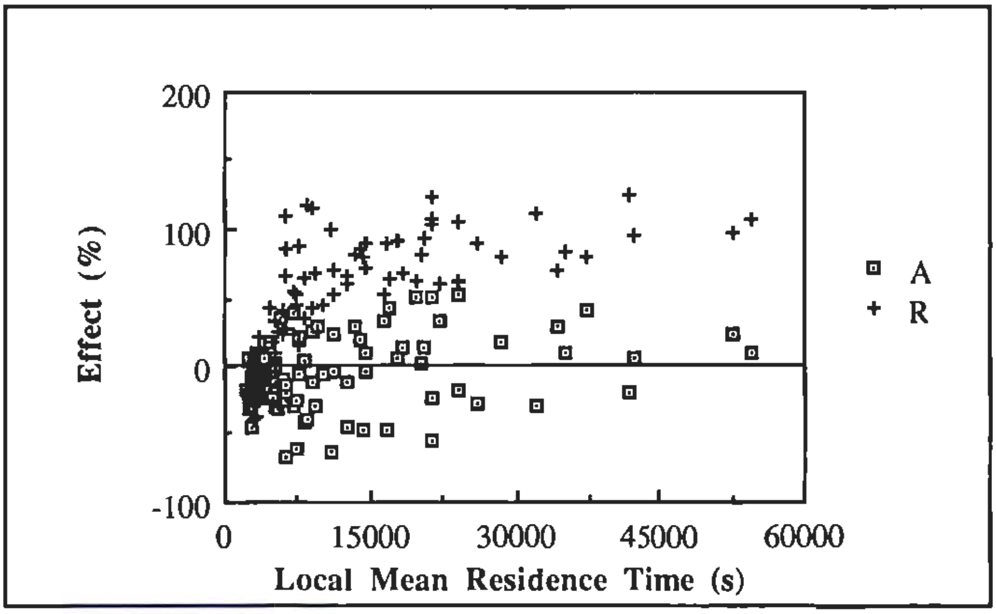
Figure 5.23: The effect of σ_a and σ_r on stage at individual cross-sections as a function of the mean stage at the cross-section for a discharge of 100 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean stage at that cross-section.

area compared to low points. However as the flow increases the cross-sectional area at narrow sections tends to decrease relative to that at shallow sections. Therefore narrow cross-sections, tend to become more important in determining the water level.

It is noted that for depth A increases only slightly with flow when expressed as a



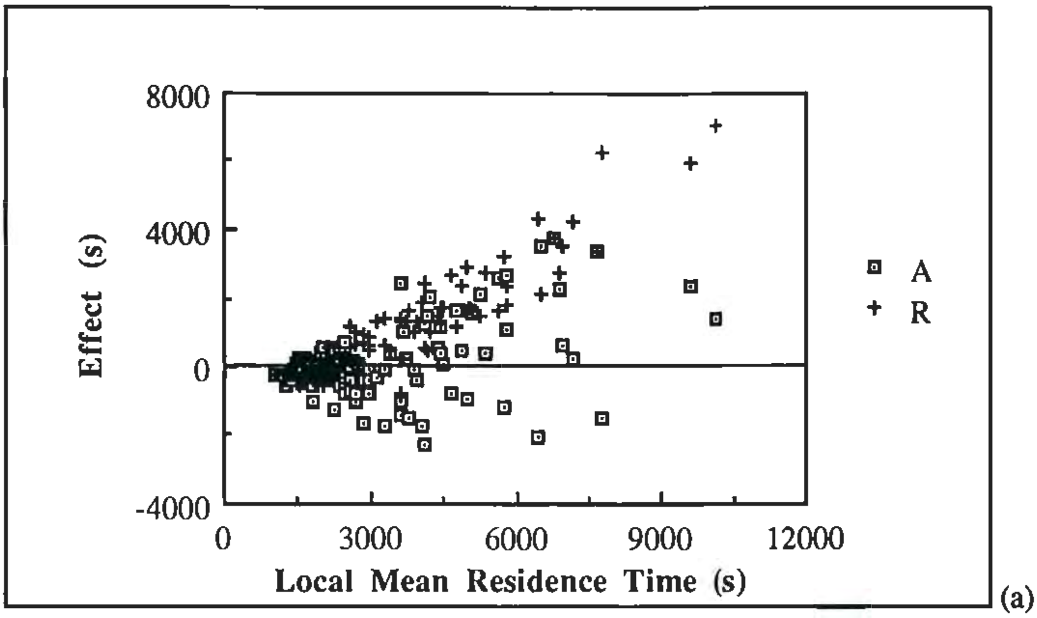
(a)



(b)

Figure 5.24: The effect of σ_a and σ_r on residence time at individual cross-sections as a function of the mean residence time at the cross-section for a discharge of 1 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean residence time at that cross-section.

percentage of the mean depth while R decreases rapidly (Figure 5.15b). The variability of a, as expressed by the coefficient of variation, does not change with the flow rate and the slight increase in A is simply due to the increasing importance of narrow cross-sections in determining the water level at high flows. The decrease in R (Figure 5.15a) as a percentage of the mean depth is due partly to the



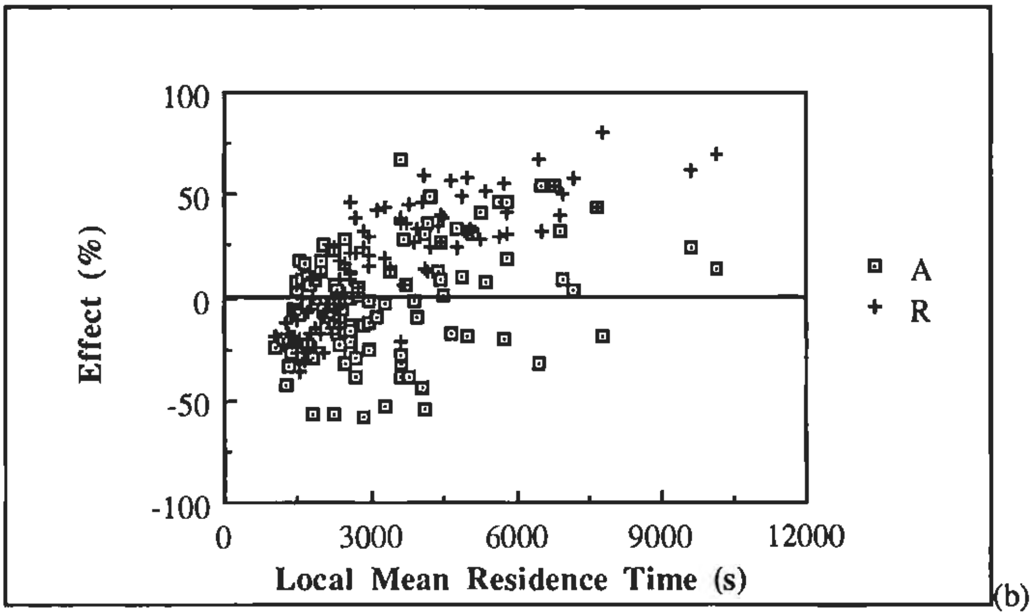
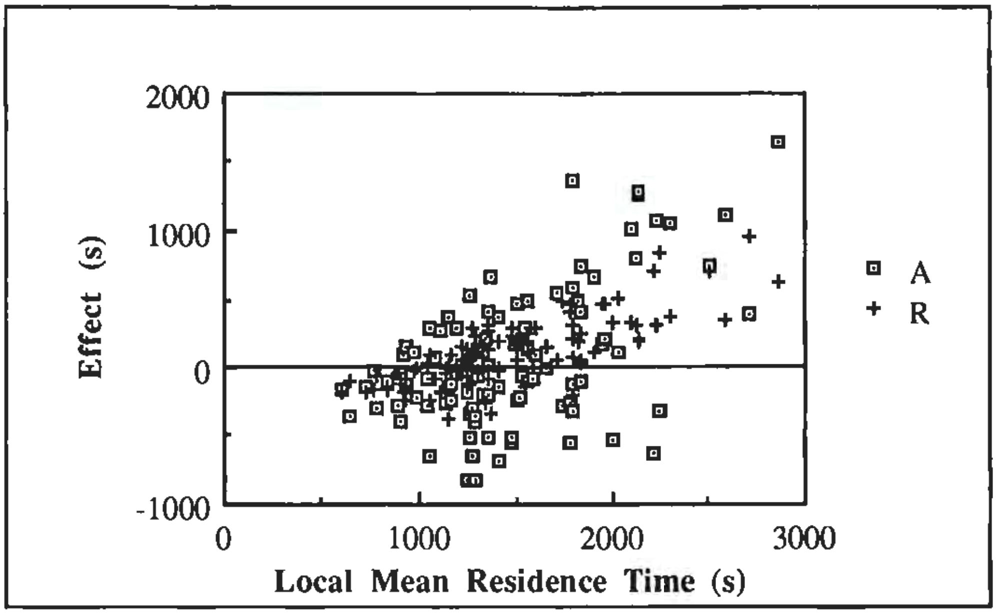
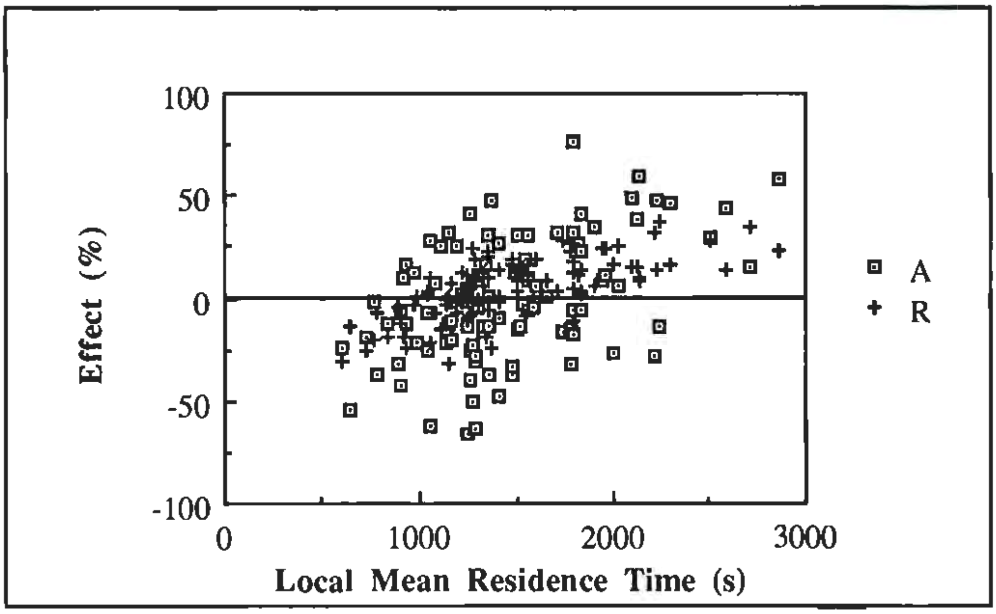


Figure 5.25: The effect of σ_a and σ_r on residence time at individual cross-sections as a function of the mean residence time at the cross-section for a discharge of 10 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean residence time at that cross-section.

decreasing importance of topographic high points in determining the depth. A more important influence is a decrease in the invert variability, as indicated by the ratio of σ_r to $\bar{y}$, as the flow increases (Table 5.7). The ratio of σ_r to $\bar{y}$ is used to indicate the invert variability because the coefficient of variation of r is undefined



(a)



(b)

Figure 5.26: The effect of σ_a and σ_r on residence time at individual cross-sections as a function of the mean residence time at the cross-section for a discharge of 100 m^3/s. (a) Absolute effect. (b) Effect as a percentage of the mean residence time at that cross-section.

since $\mu_r = 0$. $\bar{y}$ is the mean vertical extent of the wetted cross-section and σ_r is the vertical variability, so the ratio is directly analogous to a coefficient of variability.

For stage the behaviour and interpretation of A is identical to those for depth since neither the bed level nor the cease to flow level is affected by σ_a (Figures 5.21, 5.22, 5.23). The behaviour of R for stage is different from that for depth. Stage is calculated relative to the cease to flow level hence processes which determine the water level relative to the cease to flow level are important. Stage is controlled by the stage in flow constrictions and, when these flow constrictions are located at topographic high points, the stage decreases as σ_r increases because of the increase in channel slope downstream of the high point. Hence R is negative. R does not depend on the local stage because the relative change in channel slope is similar at each controlling section.

Residence time responds to changes in σ_a and σ_r in a way that is similar to the depth except that the residence time tends to be more strongly dominated by deep areas since the velocities are so low relative to velocities in shallow areas (Figures 5.24, 5.22, 5.26). In shallow areas R is negative since the depth decrease is associated with a decrease in area; however the residence time in these areas is small. In deep areas R is large and positive due to large increases in depth and cross-sectional area associated with increasing σ_r. Absolute values of A are only large in deep areas. Since a is not correlated with r, A can be positive or negative. As a result the mean value of A (Figure 5.17) is small. Absolute values A and R decrease rapidly with flow because the residence time decreases rapidly with flow everywhere. Normalised values of R decrease rapidly with flow due to the decrease in the variability of R and normalised values of A increase slightly for the same reasons as they do for depth.

5.5.3 UNSTEADY FLOW EXPERIMENTS.

An artificial flow event, for which the peak discharge was 100 m^3/s (Figure 5.27), was used to examine the effect of channel variability on flow routing. Simulations using this event for the upstream boundary condition were conducted for each of the eight different stream channels. The initial conditions were specified using the results from the steady state simulations for a discharge of 1 m^3/s. It should be noted that modelling of flow events in the Wimmera River indicates that off-stream storage may be an important factor in flow routing (Chapter 6) and that off-channel storage has been neglected in the above simulations.

5.5.3.1 Effect of Channel Variability on Routing.

Figure 5.28 shows the resultant hydrographs after routing along the first 100 km of each channel. Two characteristics that can be calculated easily and that characterise the routing are the travel time and attenuation. The travel time was

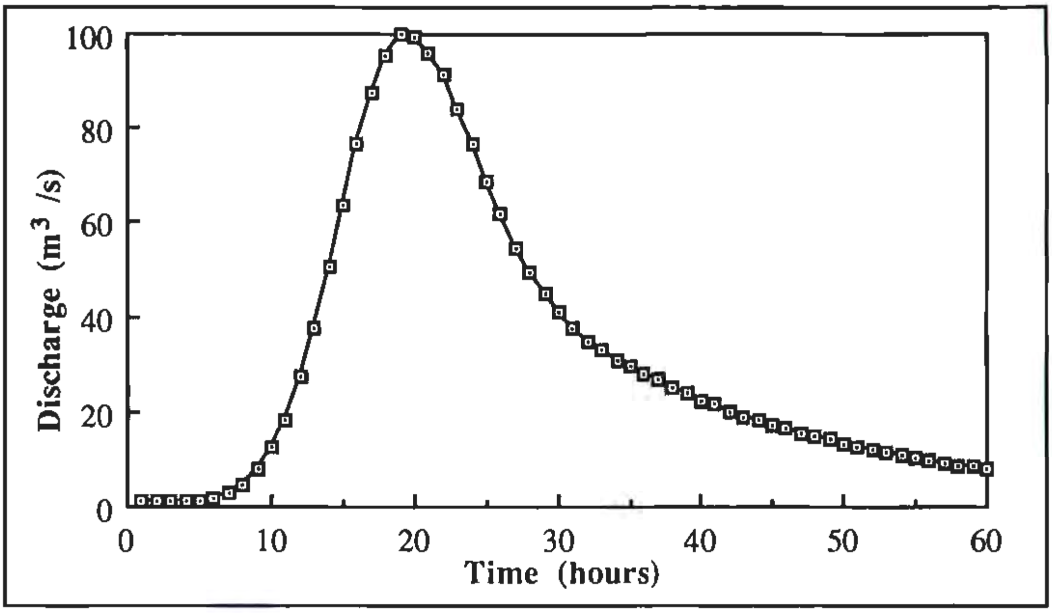


Figure 5.27: Input hydrograph used in unsteady flow experiment.

	lll	hll	lhl	llh	lhh	hlh	hhl	hhh
Travel time (hours)	35.0	38.7	35.3	37.3	37.8	40.3	37.7	39.3
Attenuation (%)	43.3	48.4	42.4	48.3	47.6	52.2	47.4	51.0

Table 5.9: Travel times and attenuation for hydrographs routed down channels of different variability.

		A	B	R	AB	AR	BR	ABR
Travel time	Effect (hours)	2.6	-0.3	2.0	-0.7	-0.4	0.0	0.0
	Effect (%)	7.0	-0.8	5.4	-1.9	-1.0	0.1	-0.1
Attenuation	Effect	4.3	-0.9	4.4	-0.2	-0.7	0.0	-0.1
	Effect (%)	9.1	-2.0	9.2	-0.4	-1.5	0.0	-0.1

Table 5.10: Effects of and interactions between different components of channel variability on travel time and attenuation.

defined as the time between the upstream and downstream peak flows. For attenuation, the difference between the upstream and downstream peak discharges normalised by the peak upstream discharge was used. Table 5.9 provides the travel times and attenuations for each simulation. These were used to calculate effects and interactions directly and are shown in Table 5.10 and Figures 5.28 and 5.29.

Figure 5.28 shows that an increase in variability generally leads to an increase in travel time and attenuation. Figure 5.29 indicates that σ_a and σ_r have the greatest

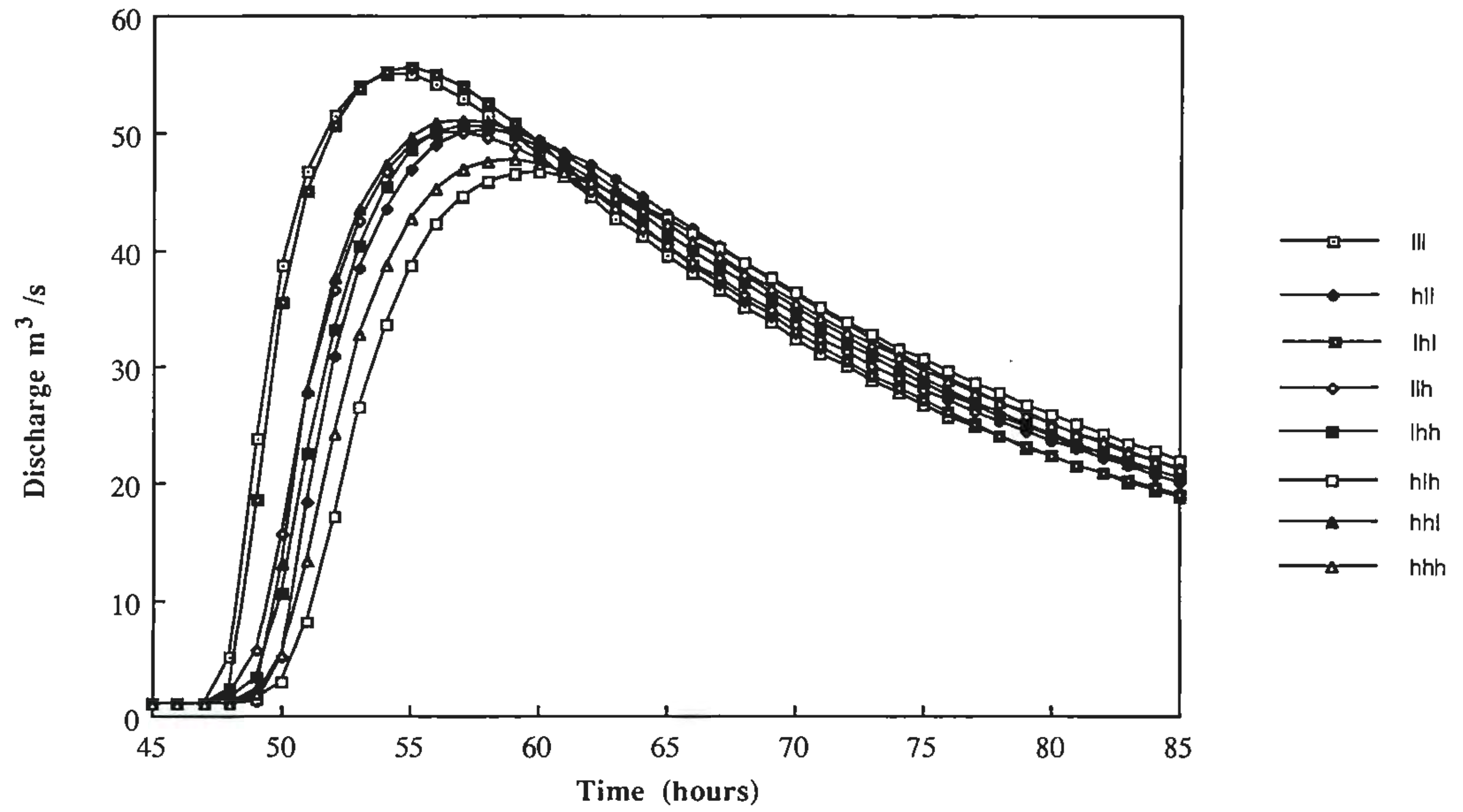


Figure 5.28: Resulting hydrographs after routing along 100 km of channel.

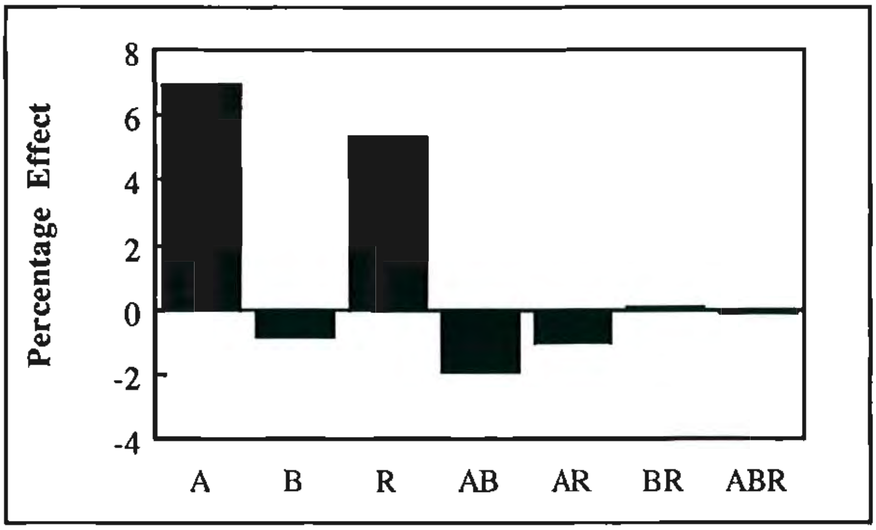


Figure 5.29: The effect of channel variability on travel time.

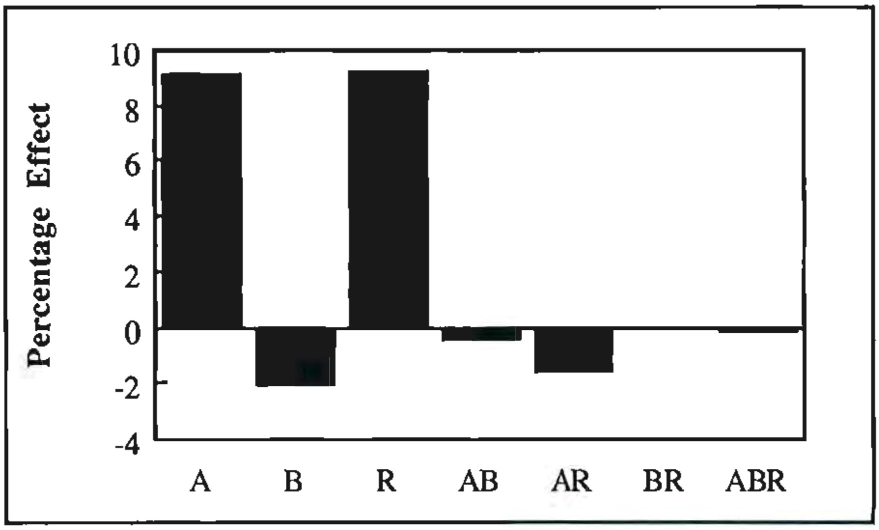


Figure 5.30: The effect of channel variability on attenuation.

impact on routing time; however the change in routing time is only 7.0% and 5.4% respectively. Given that the change in channel variability was 67%, both these effects are small. The attenuation of the peak flow is affected more by changes in σ_a and σ_r (Figure 5.30). However, both A and R are still relatively small compared to the change in variability. Neither the travel time nor the attenuation of peak discharge is sensitive to channel variability in this case. This result is consistent with the steady flow cases since neither the stage nor depth (and thus the water surface width) were sensitive to changes in channel variability.

5.5.3.2 Channel Variability vs Flow Resistance.

Given that the main parameter used in hydrodynamic models for the purposes of

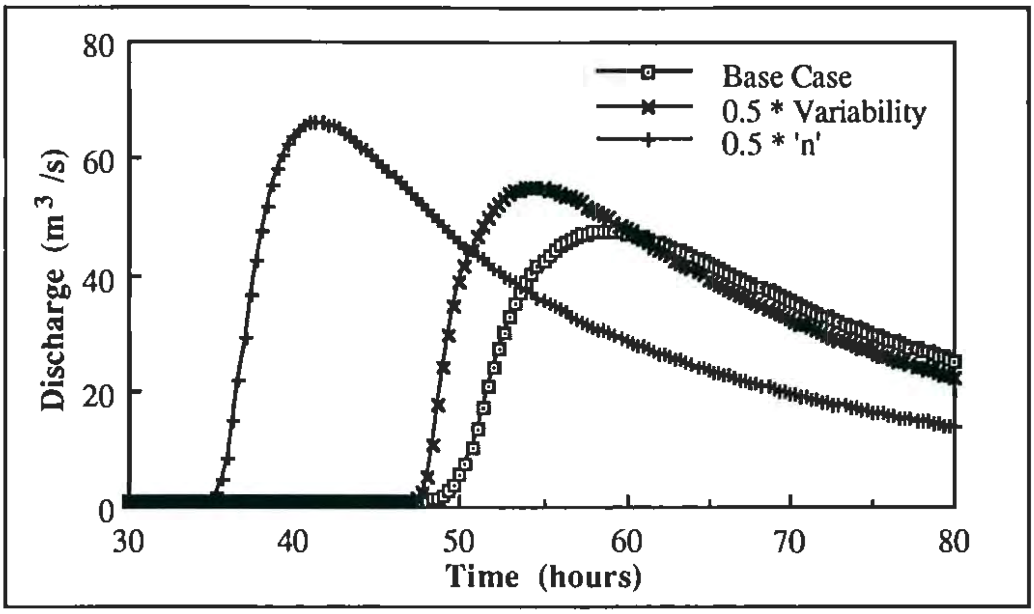


Figure 5.31: The importance of flow resistance compared with channel variability.

calibration is the flow resistance parameter it is useful to compare the impact of changing this parameter and changing the channel variability. An additional simulation was conducted in which the high variability channel, case hhh, was used and the value of the flow resistance parameter, n, was halved from 0.06 to 0.03. Figure 5.31 shows the hydrographs for the hhh case, the hhh case with n halved and the lll case. Results from these three simulations allow a comparison of the effect of halving the flow friction and halving the channel variability.

Halving the channel variability lead to an 11% decrease in travel time and a 15% increase in attenuation. On the other hand halving the flow resistance led to a decrease of 44% in the travel time and 37% decrease in attenuation. It is obvious that, in this case, the flow resistance has a significantly greater impact on hydrograph routing than the channel variability. It would therefore seem likely that errors in specifying the channel characteristics which lead to errors in channel variability and hence the flow routing behaviour would be compensated for in the calibration process.

5.5.4 IMPLICATIONS FOR MODELLING.

The above experiment demonstrates that the hydraulic characteristics of a channel typical of the lower Wimmera River are not significantly affected by channel variability except at low flows. Channel invert variability is the most important component of the channel variability and this influences, in order of increasing effect, the stage, depth and residence time. The travel time and attenuation of a hydrograph are relatively unaffected by channel variability. Perhaps the most significant conclusion is that, while the model predictions are insensitive to channel variability when averaged over a sufficiently large number of cross-sections, predictions at specific locations are sensitive to changes in cross-sections. This has important implications for model testing when predictions at specific cross-sections are of primary interest. These are discussed in Chapter 10.

If modelling of floods is of interest, the above results indicate that channel variability, for a channel of this type, is unimportant for the following reasons. The impact of channel variability is small at high flows and the routing of a hydrograph is significantly more sensitive to flow resistance than channel variability. For floods, the role of flood plain storage would act to further reduce the importance of the channel characteristics. Therefore it would seem that channel variability could be ignored. However the above analysis has concentrated on one characteristic channel and, therefore it would not be wise to make any generalisations.

On the other hand if water quality is an issue or the distribution of depth and velocity is required for a habitat study, then low flows will be of interest. In this case channel variability would be of significant importance. Again this statement cannot be applied generally; however whenever backwater effects are significant at low flows, then it can be intuitively argued that the channel invert variability would be important.

5.6 SUMMARY.

The specification of the stream channels in hydrodynamic models and the characteristics of alluvial channels were discussed briefly. Stream channel shape is specified using a series of cross-sections which must adequately represent spatial variations in the stream channel. It was argued that storage of water in the Wimmera River resulting from the presence of large pools during low flow periods was significant compared to the amount of water in moderate flow events. Therefore these pools, which result from the spatial variation in the river channel morphology, should be included in the MIKE 11 model.

Cross-sections from the Wimmera River were characterised using three parameters: a; b; and r. The variation of these parameters with channel type and along the river was then examined. A statistically significant difference between cross-sections from single channel and anabranching reaches of the Wimmera River was found. This variation occurs at a spatial scale that is significantly greater than the typical meander length. Other findings are summarised in §5.3.3. Using the analysis of available cross-sections from the Wimmera River as a basis, a stochastic cross-section model was developed and used to infill cross-sections for the MIKE 11 model of the river.

A numerical experiment was conducted to assess the importance of channel variability in determining the hydraulic characteristics of an open channel which is nominally similar to the lower Wimmera River channel. This experiment included three different steady flow rates and one unsteady flow. When the steady flow response was averaged over 100 cross-sections, the channel variability was significant only for low flows. Residence time was more sensitive than stage or depth to changes in channel variability. In contrast to the mean response, predictions at specific cross-sections were sensitive to changes in the cross-sectional characteristics. The effect of channel variability on the routing of hydrographs was small compared with the effect of flow resistance. The results from the numerical experiment indicate that inclusion of channel variability is important for low flow simulations and where predictions at specific cross-sections are of interest.

CHAPTER 6 - ONE-DIMENSIONAL MODELLING OF THE WIMMERA RIVER.

6.1 INTRODUCTION.

A one-dimensional model of the Wimmera River between Glynwylln and Lochiel has been developed and is described below. MIKE 11 (DHI, 1992a, 1992b), which is a physically-based numerical river modelling package, was used. The diffusive wave approximation to the St Venant equations and the Advection Dispersion equation form the basis of the model developed. The primary aim of this exercise was to develop a solute transport model which is applicable to in-bank flows. Therefore no attempt to model the interaction between the river and flood plain was made. While the model is capable of continuing simulations through a major flood, the results of any such simulation would obviously not be reliable. At the completion of this study the model is to be used to investigate the effects of different flow management regimes on river salinities and saline pools.

This chapter deals with the application of MIKE 11 to the Wimmera River and is limited to a consideration of the 1-Dimensional model. It therefore excludes saline pools. Descriptions of the model structure, additional algorithms written to simulate specific features of the system and to obtain additional summary results, and the model calibration and testing are included. Discussions of model results, modelling difficulties and model appropriateness are also included. This 1-dimensional model is extended to incorporate saline pools in Chapter 9.

6.2 THE MODEL STRUCTURE.

This section discusses the model structure, modelling of specific features of the Wimmera River system and additional algorithms incorporated in the model.

The physical system being modelled is specified in MIKE 11 in a hierarchical manner. Firstly the channel network is defined, then cross-sections and hydraulic structures are added and the location of flows to and from the model specified. In this application of the model, inflows and outflows are specified by using time-series of discharge (see Chapter 4) or stage-discharge relationships.

Of course any modelling involves idealisation in the description of processes and in the physical definition of the system due to imperfect knowledge. The significant idealisations of the physical definition of the Wimmera River system are discussed below. Idealisations inherent in the description of processes were discussed in Chapter 3.

6.2.1 CHANNEL NETWORK, CROSS-SECTIONS AND LATERAL INFLOWS.

The channel network is defined by determining the lengths of all the channel reaches and connections between reaches. Figure 6.1 shows the reaches used for the model of the Wimmera River. Cross-sections were specified in each reach at an interval of approximately 1 000 m. Wherever measured cross-sections were available they were used unless two within 500 m of each other were available in which case one was chosen. Where the spacing of available cross-sections exceeded 1 500 m additional, stochastically-generated, cross-sections were added so that a spacing as close to 1 000 m as possible resulted. The method used to generate these cross-sections is discussed in Chapter 5.

The location of lateral inflows from sub-catchments adjacent to the river were included in the model as follows. Topographic maps (1:100 000 scale) were examined and significant tributaries were identified. Inflows to the river were then specified at those points as time-series. It was assumed that the entire contribution of the catchment adjacent to the river was concentrated at these points which are shown on Figure 6.1. The method used to derive each time-series is described in detail in Chapter 4.

6.2.2 WEIRS.

Several weirs exist along the Wimmera River. Major weirs are Glenorchy Weir, Huddlestons Weir, Ashen's Weir, Horsham Weir and Dimboola Weir and minor weirs associated with stream gauges exist at Glynwylln, Glenorchy and downstream of Horsham. The major weirs are shown on Figure 6.1 All the weirs except Horsham and Dimboola Weirs are broadcrested weirs and have been modelled as such.

The spacing of cross-sections about a weir involves a compromise. Flow friction is ignored between the cross-sections separated by a weir (DHI, 1992b) therefore cross-sections should be placed relatively close to the weir. However it is desirable to maintain advective Courant Numbers less than unity in the solute transport simulations, so placing cross-sections further from the weirs is an advantage since it allows longer simulation time-steps. There is also an improvement in the stability of the hydrodynamic simulations at some weirs when a larger spacing is used. A satisfactory compromise was reached by including

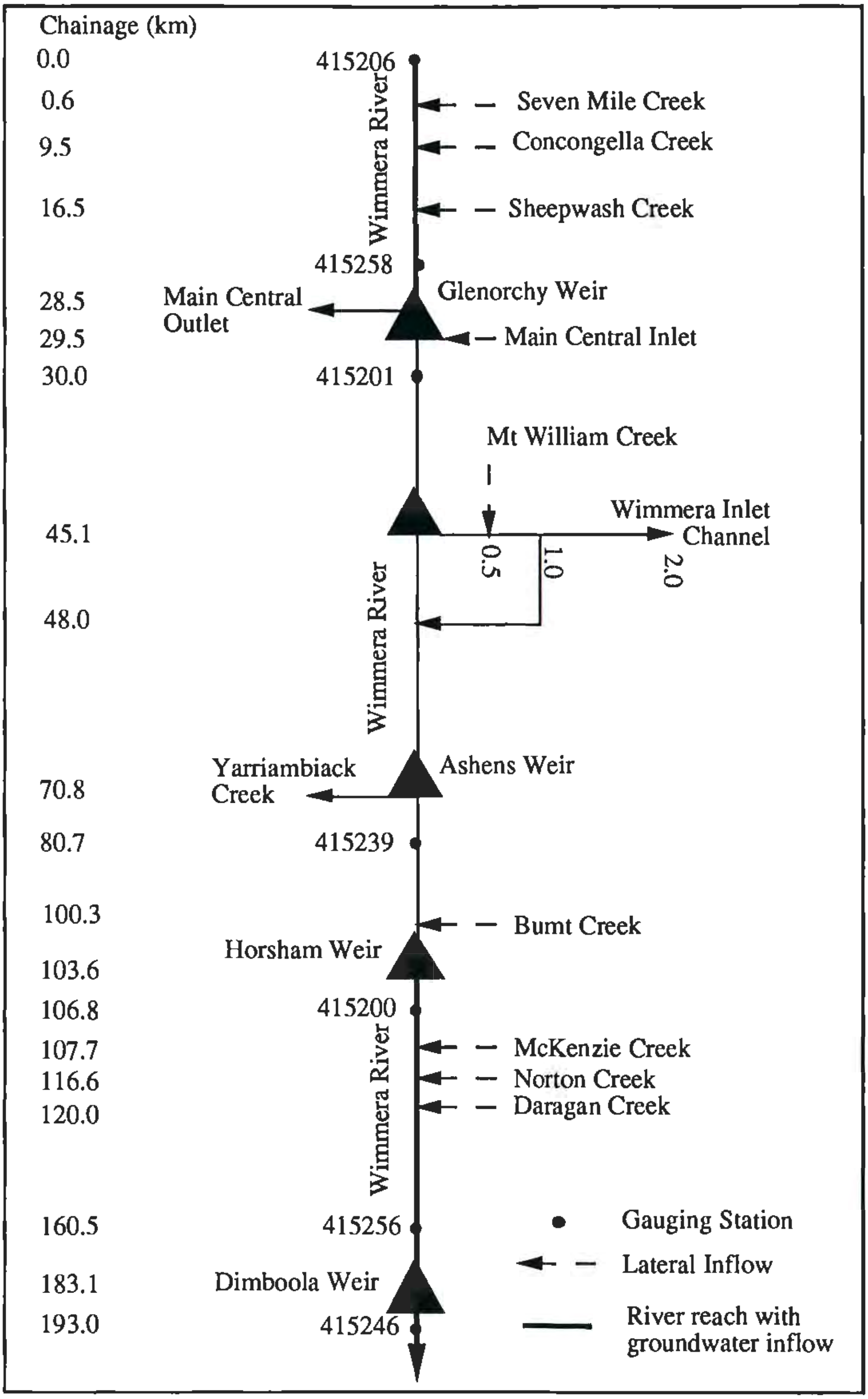


Figure 6.1: Model network diagram showing the location of channel reaches, lateral inflows, groundwater inflows, gauging stations and major weirs.

cross-sections at points 100 m upstream and downstream of each weir. The downstream cross-section was simply a copy of the upstream one.

Horsham and Dimboola Weirs are drop-board structures; however they were modelled as broadcrested weirs. This was done for the following reasons. The operation of these structures during floods is determined locally on a flood-by-flood basis and is therefore not very amenable to a generalised mathematical description. During smaller events no change is made to the regulation at the weirs which then act as broad crested weirs anyway. Also there is no routine in MIKE 11 capable of directly modelling drop-board structures. Apart from water levels immediately upstream of these weirs, this idealisation will only have a small influence on model predictions because of the relatively small storage capacity of each weir pool.

6.2.3 GLENORCHY WEIR POOL.

The Glenorchy Weir and associated facilities is an important part of the Wimmera River System since interaction between the Wimmera Mallee Stock and Domestic Water Supply System (WMSDS) and the river occurs there. Figure 6.2 shows the Glenorchy Weir Pool and the channels entering and leaving the pool. The inflowing channel is modelled by specifying an inflow time-series at the first cross-section upstream of the weir. The outflowing channel was modelled by specifying a channel which flows out of the river at the second cross-section upstream of the Glenorchy Weir. The flow in this channel is specified by a daily time-series obtained from data recorded for the operation of the WMSDS. An arbitrary stage-discharge relationship was then specified at the downstream end of this short channel so that water can leave the model.

The above specification of the outflowing channel was necessary so that water could be removed from the model at the salinity predicted by the transport simulations. A time-series specifying a lateral outflow from the main river channel was not used as the outflow of salt from the model would be incorrectly modelled. This is because the salinity of a lateral outflow is specified using an externally supplied time-series rather than by using the predicted river salinity at the lateral outflow.

6.2.4 HUDDLESTONS WEIR.

6.2.4.1 The Physical system.

A major diversion from the Wimmera River into the Wimmera Inlet Channel occurs at Huddlestons Weir (Figure 6.3). The Mt William Creek enters the

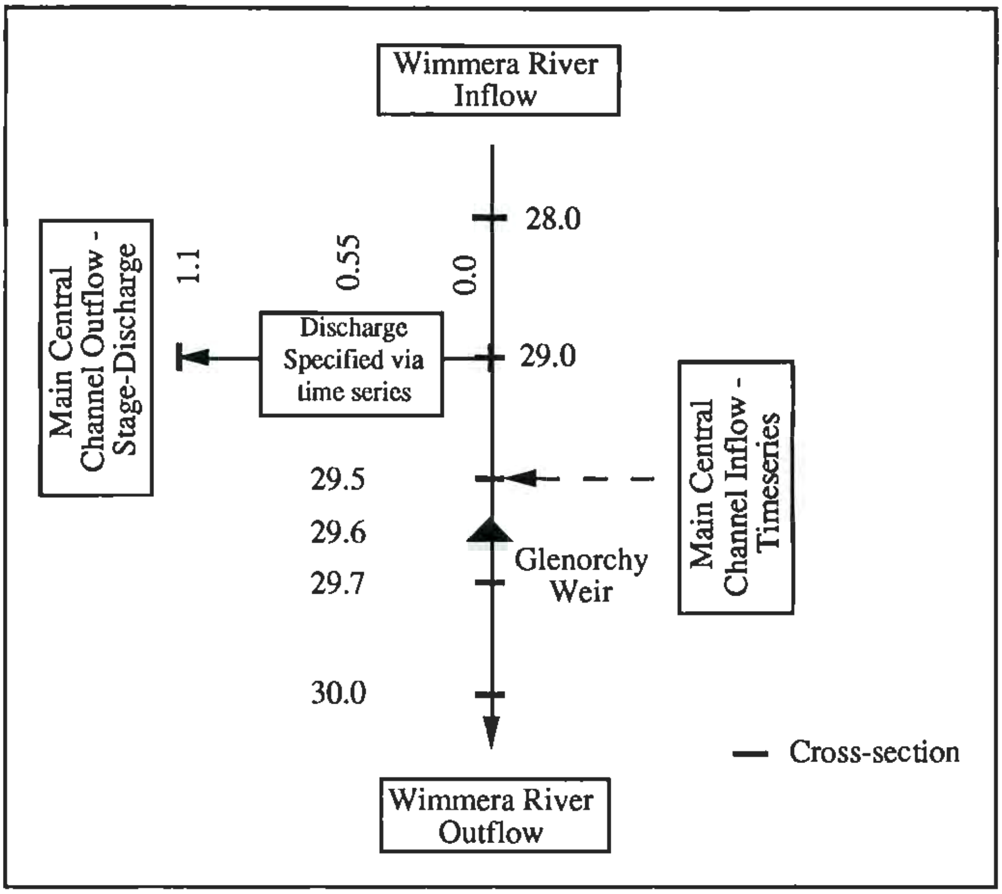


Figure 6.2: Representation of the Glenorchy Weir Pool in the model.

Wimmera Inlet Channel downstream of this diversion. Mt William Creek has two channels: one that carries both low and high flows (low flow channel) and one that carries high flows only (high flow channel). The discharge in the high flow channel is significant only when the discharge in the Mt William Creek is very high. It is estimated that 2 000 Ml/d is carried in the high flow channel when the total flow in the Mt William Creek reaches 10 000 Ml/d (R. McIlvena, RWC, Per. Com.). Small flows also enter the Wimmera Inlet Channel from Golton Creek and a number of other small tributaries.

All the flows from these different tributaries were simply lumped into one and included in the model as a lateral inflow. This was located at the point where the low flow channel of the Mt William Creek enters the Wimmera Inlet channel. Neither the flows in the Mt William Creek nor the flows in the other tributaries are gauged. Estimation of these flows was discussed in Chapter 4.

Along the Wimmera Inlet Channel a number of emergency release structures exist. These provide protection against flood flows which could exceed the channel capacity. Water spilt from these structures eventually finds its way back

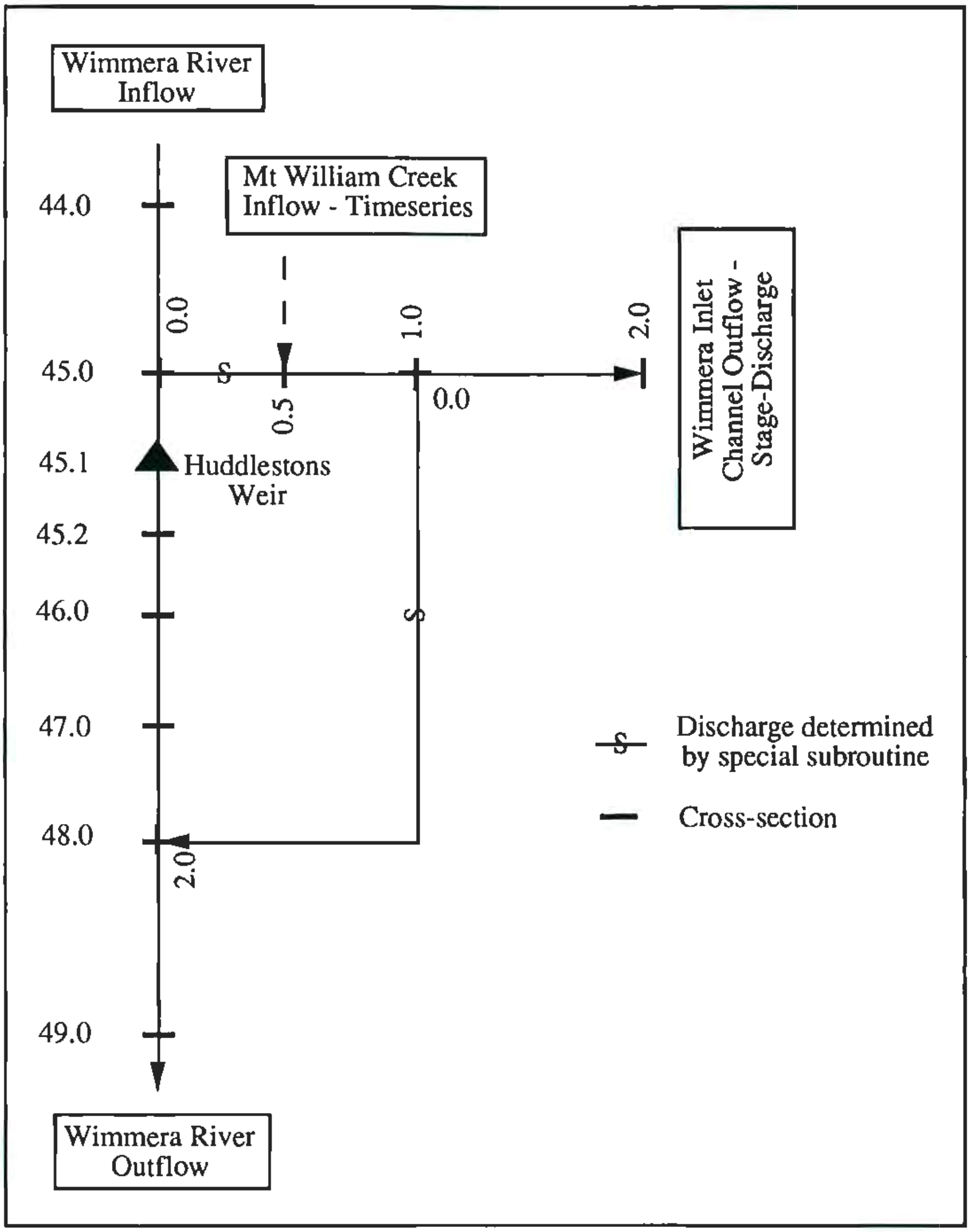


Figure 6.3: Representation of Huddlestons Weir and the Wimmera Inlet Channel in the model.

into the Wimmera River through a series of channels which join the Wimmera River at various points over the next 20 km. The most significant of these structures are a 500 m long side spill weir located downstream of the realigned Mt William Creek inflow and a sluice gate at the point where the high flow channel of the Mt William Creek crosses the Wimmera Inlet Channel (Barlow, 1987).

The spills from the Wimmera Inlet Channel were modelled by assuming that they all occurred at the side-spill weir and that the spilt water was returned back to the

Wimmera River a short distance downstream of Huddlestons Weir. In other words, all the relief structures are lumped together and the various channels through which water returns to the Wimmera River are replaced by a short channel section. Downstream of Huddlestons Weir the Wimmera River consists of a number of anabranches the modelling of which is discussed in §6.2.6.

Flows of water to and from the Wimmera Inlet Channel are controlled by a number of structures which are operated by WMSDS personnel. Within the model the distribution of flows to and from the Wimmera Inlet Channel is specified by a subroutine written specifically for the purpose.

6.2.4.2 Operation Procedures and Model Algorithms.

Diversions from the Wimmera River at Huddlestons Weir are ultimately to Pine and Taylors Lakes. Typically the entire flow in the Wimmera River is diverted into the Wimmera Inlet Channel unless; the channel capacity is insufficient, in which case the channel is operated at full capacity. If the lakes are full the diversion is closed down. However the diversion may be closed on some other occasions; for example during the first flush from the catchment which is typically more saline than average (R. McIlvena, RWC, Per. Com.).

Figure 6.4 shows a flow chart for the subroutine which simulates this diversion and any spill from the Wimmera Inlet Channel. Periods when the diversion was not operating were determined before a simulation by examining the storage records for the lakes along with the historical flow in the Wimmera Inlet Channel and the Wimmera River at Glenorchy. If the Lakes were at full capacity or if there were a significant flow recorded in the Wimmera River, but not in the Wimmera Inlet Channel, then the diversion was closed. Otherwise the diversion was open. This information was supplied to the model using a binary time-series.

During a simulation the diversion is calculated as follows. The flow in Mt William Creek, Q_{mtw}, and Wimmera River, Q_{wr}, are determined. The maximum allowable flow in the channel, Q_{max}, is set to the channel capacity if the diversion is open or zero if the diversion was closed. A linear interpolation over 36 model time-steps was used between these two values whenever the diversion was opened or closed. The maximum diversion from the Wimmera River Q_{dmax}, was then calculated using Equation 6.1.

$$Q_{dmax} = Q_{max} - Q_{mtw} \qquad \text{for } Q_{max} - Q_{mtw} > 0 \qquad (6.1a)$$

$$Q_{dmax} = 0 \qquad \text{for } Q_{max} - Q_{mtw} \leq 0 \qquad (6.1b)$$

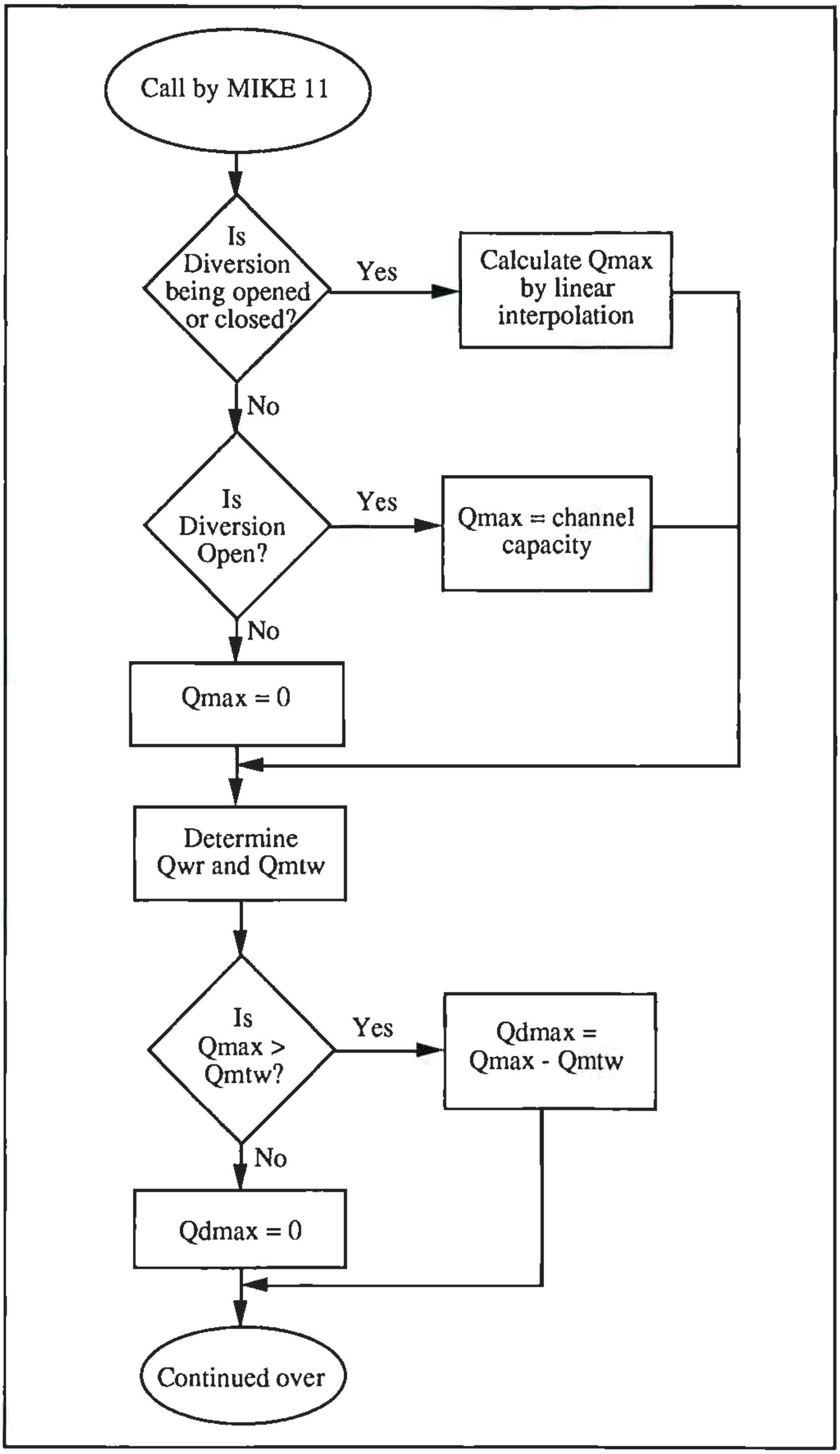


Figure 6.4: Flow chart describing the modelling of the Huddlestons Weir Diversion.

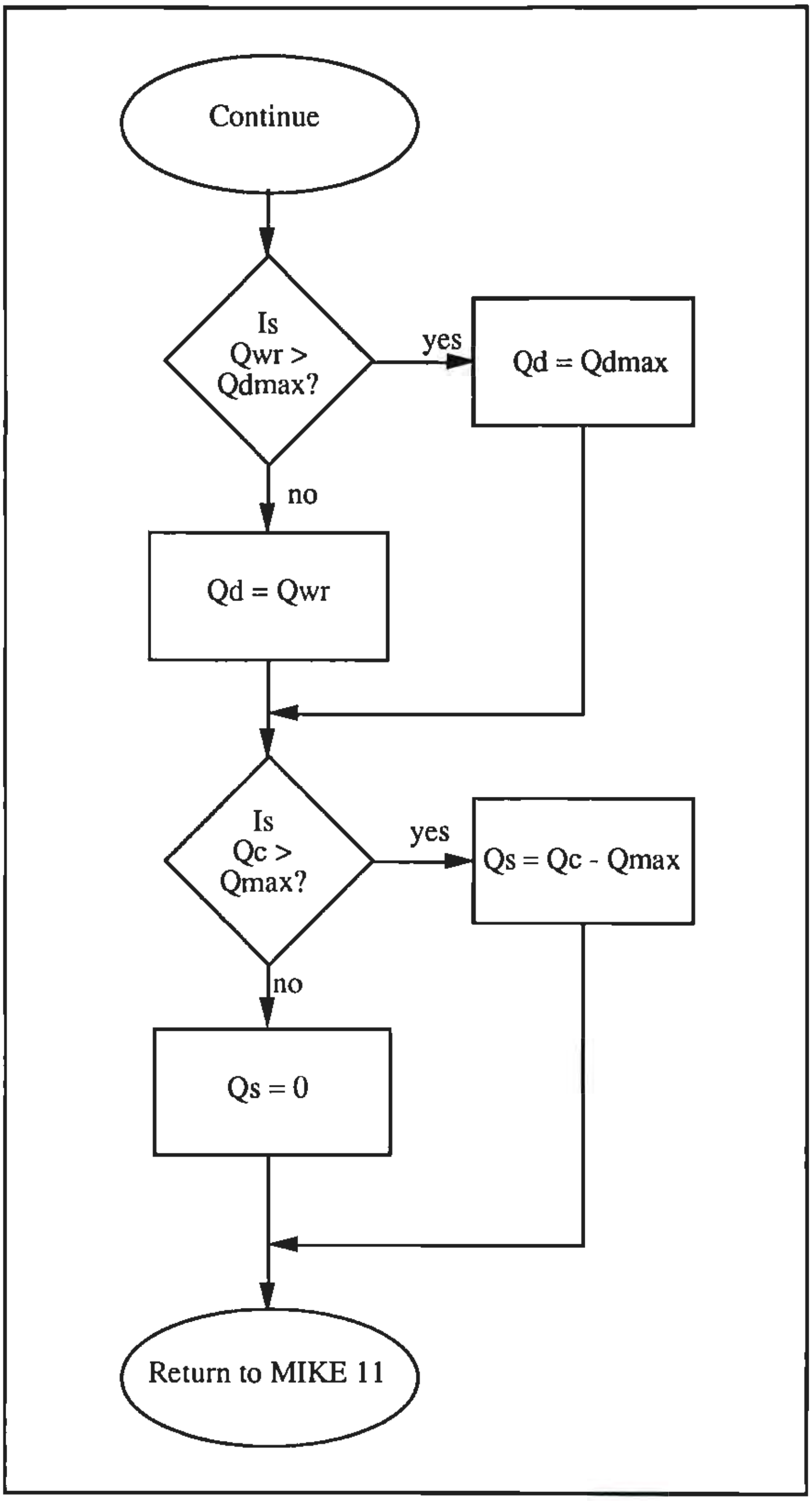


Figure 6.4 cont': Flow chart describing the modelling of the Huddlestons Weir Diversion.

The diversion from the Wimmera River , Q_d, was then calculated as the smaller of the Q_{dmax} and Q_{wr}.

Spillage, Q_s, from the Wimmera Inlet Channel occurs if the flow in the channel, Q_c, downstream of the Mt William Creek inflow is greater than Q_{max}. Q_s is calculated using Equation 6.2.

$$Q_s = Q_c - Q_{max} \qquad \text{for } Q_c - Q_{max} > 0 \qquad (6.2a)$$

$$Q_s = 0 \qquad \text{for } Q_c - Q_{max} \leq 0 \qquad (6.2b)$$

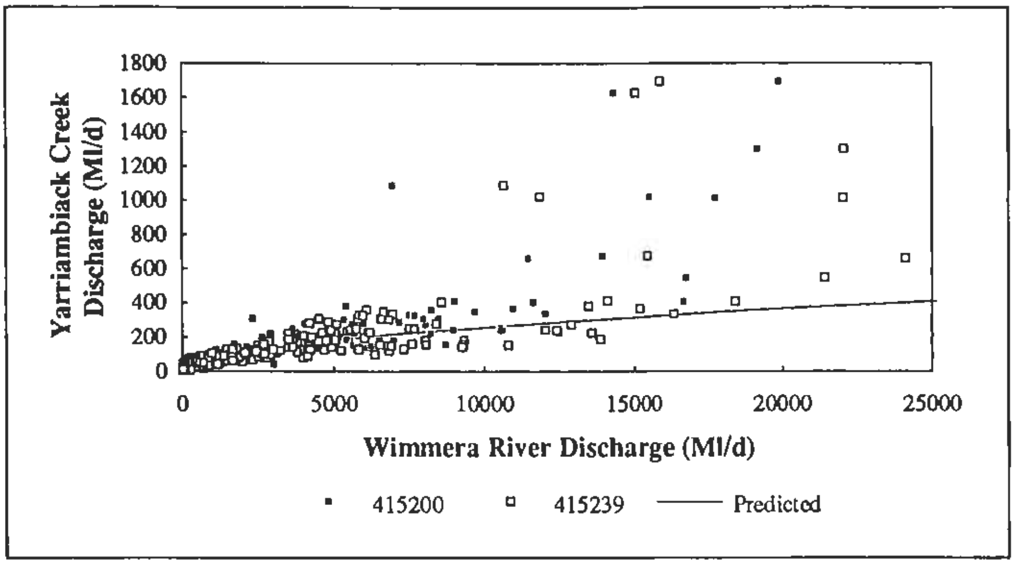
Spills only occur if the flow in the Mt William Creek is greater than the maximum allowable flow in the channel. When the diversion is closed, the entire flow in the channel downstream of Mt William Creek is spilt.

The modelling of water distribution in the vicinity of Huddlestons Weir and the Wimmera Inlet Channel is one of the most significant conceptualisations and simplifications of the Wimmera system in the model. It does however reflect the general operation of this part of the system.

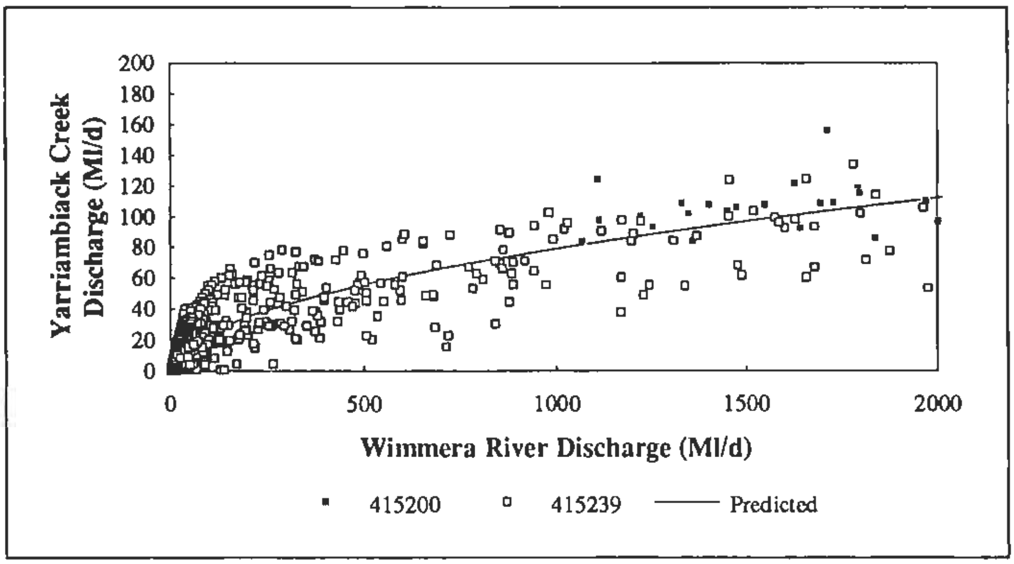
6.2.5 YARRIAMBIACK CREEK.

Another outflow from the Wimmera River occurs at Yarriambiack Creek which is a distributary stream. During significant floods, some of the water leaving the Wimmera at the Yarriambiack Creek returns further downstream; however since floods are of limited interest this has been ignored. This outflow from the model has been included by specifying a short channel leaving the river; the flow into which is controlled by a subroutine written for the purpose.

The Yarriambiack Creek outflow, Q_y, is determined as a function of the flow in the Wimmera River, Q_w, upstream of the Yarriambiack Creek. This function was established by regressing Q_y with Q_w. Since Q_w is not gauged it was estimated by adding the flow in the Wimmera River at Drung Drung to Q_y and the flow in the Wimmera River at Horsham to Q_y. The River flows were lagged by one day because an examination of the hydrographs indicated there was approximately one days difference in the timing of the peak flows. Although the Drung Drung gauge is closer to Yarriambiack Creek than the Horsham Gauge and is therefore preferable, it is only a flood gauge and is not rated for low flows. Therefore the Horsham Gauge was also used.



(a)



(b)

Figure 6.5: Relationship between the discharge in the Wimmera River upstream of Yarriambiack Creek and the discharge in the Yarriambiack Creek.

A linear relationship between flow in the Wimmera River upstream of Yarriambiack Creek and the flow in Yarriambiack Creek was used for low flows and a power curve was used for higher flows. A linear weighting of the two was used about their intersection point, thus providing a smooth transition between the two. Figure 6.5 shows the relationship between the discharge in the Wimmera River and Yarriambiack Creek and the relationship used to model the flow in Yarriambiack Creek. This relationships is:

$$Q_y = 0.368\ Q_W \qquad Q_W < 34.56 \qquad (6.3a)$$

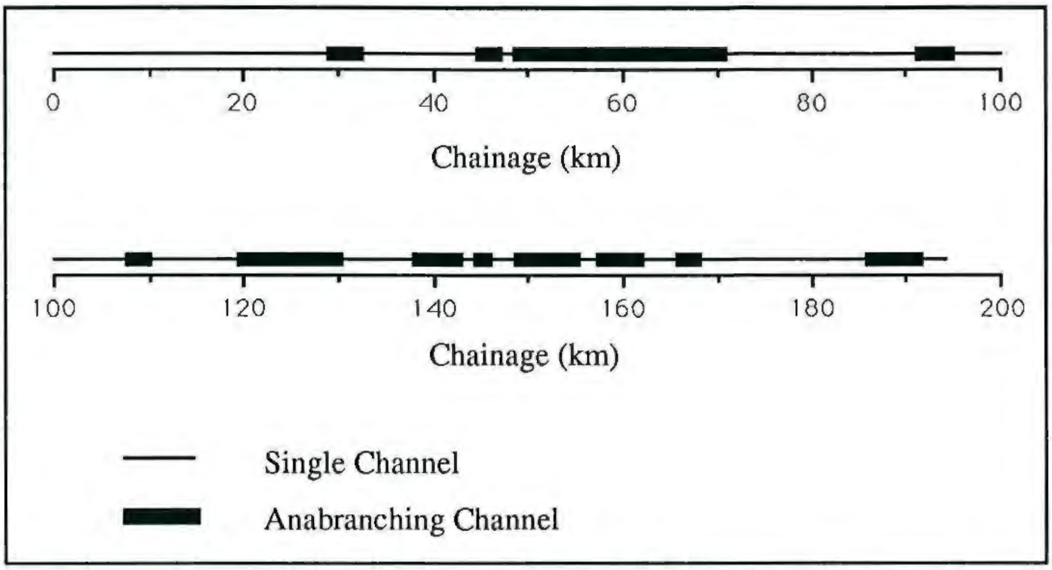


Figure 6.6: Distribution of anabranching channel sections along the Wimmera river between Glynwylln and Lochiel. Chainage increases downstream from Glynwylln gauging station.

$$Q_y = 0.368\left(\frac{51.84 - Q_w}{17.28}\right) Q_w + 2.329\left(\frac{Q_w - 34.56}{17.28}\right) Q_w^{0.51} \qquad 34.56 \le Q_w < 51.84 \qquad (6.3b)$$

$$Q_y = 2.329\, Q_w^{0.51} \qquad Q_w \ge 51.84\,. \qquad (6.3c)$$

In Equation 6.3, Q_w is the discharge in the Wimmera River upstream of Yarriambiack Creek and Q_y is the discharge in the Yarriambiack Creek. All discharge units are Ml/d.

6.2.6 ANABRANCHING CHANNELS.

Sections of anabranching channels are a common feature of the Wimmera River (Figure 6.6). These anabranches were referred to as wetland areas in the analysis of cross-section data carried out in Chapter 5. Typically a number of relatively small, shallow channels flow through these areas which are relatively low lying. No detailed survey data are available relating to the channel network or flood plain in these areas; however a small number of cross-sections are available. It was therefore necessary to devise a simplified representation of these sections of the river.

It was possible to estimate the number of channels, excluding short channels linking two long anabranches, flowing through these areas from topographic map information (1:100 000). This information was then used to specify an effective channel with hydraulic characteristics obtained as follows. A cross-section was obtained either from existing data or by stochastic generation. The cross-sectional area, and thus the conveyance, and the storage width were multiplied by the number of channels that flowed through the area.

This can be justified as follows. The elevation of each anabranching channel is the same at the upstream and downstream ends of one of these sections. Therefore, if they are approximately the same length as is usually the case; their mean slopes must also be similar. From the point of view of routing of water through these areas, the conveyances and storage widths of parallel channels can be added and the effective channel should have similar storage-discharge characteristics as a network model.

This treatment of anabranching channel sections is a major simplification and conceptualisation of the physical system. It assumes that it is possible to represent adequately the storage discharge characteristics of these sections with an effective channel. Inundation of the low lying parts of these areas is also ignored. It is probable that this is the least adequate part of the representation of the Wimmera River System. However without a large amount of field data collection, this cannot be improved.

6.2.7 SIMULATION OF LOW FLOWS.

The simulation of low flows is a problem with MIKE 11. When depths become small in hydrodynamic models numerical stability problems can arise (Cunge et al., 1980). MIKE 11 deals with this problem in two ways. Firstly, a slot is added to the bottom of a cross-section. This artificially increases the depth at low flows and improves the stability of the simulation. Secondly, if this slot dries out, as can happen when flows approach zero, MIKE 11 adds water to keep the solution running (DHI, 1992b). If the volume of water added is significant, this causes problems when modelling water quality because the additional discharge transports solute downstream and the added water reduces the concentrations by dilution. In the case of the Wimmera River up to 400 Ml/d of water was being added which severely compromised the low flow water quality simulations.

To overcome this problem the following modifications were made to the channel specification. Points that appeared, on the basis of the simulated water surface profile, to be producing significant backwater effects were identified (Figure 6.7). These are not hydraulic controls in the strict sense of the term since the flow downstream of these locations is still subcritical; however they are points which play an especially significant role in determining the upstream depth. They are typically the high points in the bed which lead to backwater effects and pools in

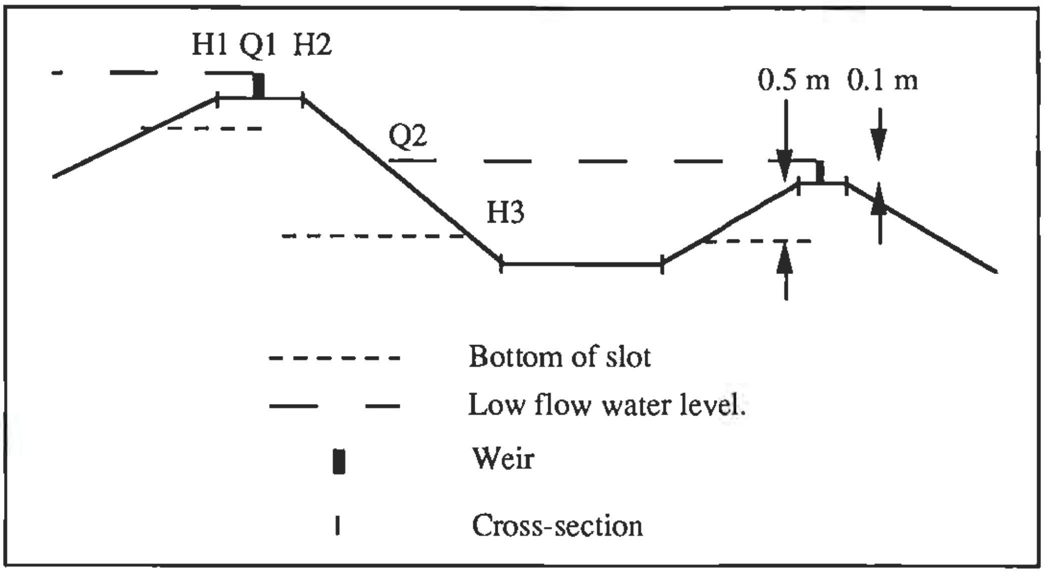


Figure 6.7: Weirs and slot used to enable the modelling of low flows.

the river. At these points a duplicate cross-section was added 200 m downstream of the controlling cross-section and a weir that was 0.10 m narrower than the channel and had a minimum crest level 0.10 m above the channel invert was added between the two cross-sections (Figure 6.7). A slot was added to all cross-sections with an invert level greater than the downstream cease to flow level minus 0.5 m (Figure 6.7).

In theory this overcomes the low flow problem because the flow is described using a weir equation at these locations rather than the diffusive wave equation and the backwaters induced by the weirs ensure that small depths do not occur elsewhere. A minimal head loss occurs over the weirs when the river is flowing at higher levels for the following reasons. During drowned flow the head loss over the weir, ΔH, is calculated using (DHI, 1992b):

$$\Delta H = \left[\zeta_{in}\left(1 - \frac{A_s}{A_1}\right) + \zeta_{out}\left(1 - \frac{A_s}{A_2}\right)^2\right] \frac{1}{2g}\left(\frac{Q_s}{A_s}\right)^2, \qquad (6.4)$$

where A_1, A_s and A_2 are the cross-sectional areas upstream of the weir, at the weir and downstream of the weir respectively, Q_s is the discharge through the weir and ζ_{in} and ζ_{out} are inflow and outflow loss coefficients. Since the weirs are only marginally smaller than the cross-sections themselves the energy losses are small. Flow simulations including and excluding these weirs showed that the weirs did not affect significantly the routing of flow events.

In practice this method of overcoming the low flow problems was difficult and a significant amount of time was spent adjusting the cross-sections and weirs.

These difficulties are now discussed. Firstly, problems can occur as the discharge increases from low values. During low flows the water level at the cross-section immediately downstream of these weirs, H2, tends to fall down into the slot while the water level at the second cross-section downstream, H3, may be above the slot (Figure 6.7). Because the conveyance at Q2 is calculated using the conveyances at both H2 and H3, water can be rapidly removed from H2 even if the water level at H2 is low. In some cases this means that the water level at H2 will not rise out of the slot and $A_s >> A_2$. This in turn means that the head loss across the weir will become large and the flow through the weir will be significantly smaller than it should be. The upstream level, H1, will then rise until the simulation is stopped in an error condition because the upstream water level becomes too high or the water level at H2 eventually moves above the top of the slot at which time A_2 increases rapidly, the head loss across the weir drops and a surge moves downstream.

This problem could generally be overcome by either setting ζ_{out} to zero or by increasing the cross-sectional area and decreasing the hydraulic radius for the slot in cross-section H2. By doing this the conveyance through the cross-section was kept relatively small and the increase in cross-sectional area meant that the head loss across the weir was smaller and the discharge through the weir larger. Thus the water level at H2 rose more rapidly and the problem was avoided. If this did not fix the problem then the level at which slot ended and the cross-section began had to be reduced so that the water level at H2 moved above the top of the slot earlier.

These weirs also tended to cause oscillations in water levels adjacent to the weir and in discharge through the weir. These oscillations typically had a period that was twice the simulation time-step. These were reduced in a number of ways. The first involved setting the time-weighting in the finite difference scheme fully forward in time. This introduces some numerical diffusion into the scheme and tends to attenuate any high frequency waves such as the oscillations at the weirs. Provided any changes in discharge caused by floods occur over a much longer period than the simulation time-step, as is the case in the model of the Wimmera River, the error introduced into the simulation by weighting the finite difference scheme forward in time is small (DHI, 1992b).

Where oscillations occurred at low flows it was also possible, in some cases, to reduce these by changing the downstream cross-section. Typically this may involve increasing the area of the slot and reducing the hydraulic radius and, in some cases, reducing the level of the top of the slot. Occasionally it was possible to improve the stability by increasing the energy loss coefficient for inflows to the weir. Sometimes, when the channel downstream of the weir was particularly steep, none of the above solutions worked and it was necessary to add another additional cross-section 400 m downstream of the weir. This was a copy of the weir cross-section but was lowered an appropriate amount so that the channel slope downstream of the weir was significantly reduced.

6.2.8 OBTAINING DAILY DISCHARGES.

Much of the flow data available for the WMSDS is in the form of daily totals. To obtain similar data from the model simulations a subroutine was written which integrated the instantaneous flow over each day at any specified location in the model.

6.3. CALIBRATING THE HYDRODYNAMIC MODEL.

6.3.1 FLOW RESISTANCE.

The flow friction parameter, n, in MIKE 11 can be varied along the channel and with flow depth and time. This allows a complex parameterisation of resistance; however the amount of information required for calibration is correspondingly large.

6.3.1.1 Some Thoughts on Calibration Data.

If data available at flow gauging stations are to be used to calibrate flow resistance, there are three types of data available: stage time-series, discharge time-series and rating curves. The choice of which of these data sets to use when calibrating the flow friction parameter is not necessarily clear because the different data sets represent different processes.

Flow in natural streams like the Wimmera River is nearly always subcritical (Henderson, 1966). Therefore, for a given discharge, the observed stage represents the effect of the downstream hydraulic control and the flow resistance between the hydraulic control and the point of observation. In irregular channels classic hydraulic controls are often difficult to identify. However from the point of view of this discussion the downstream hydraulic control can be thought of as a combination of those points where the channel is constricted by a narrowing or by the bed rising. These points are especially significant in producing backwater effects and therefore the observed stage.

On the other hand the discharge observed at a point in a stream results from inflows to the stream combined with the routing processes operating upstream of the point of observation. The routing process in natural streams is dominated by the transient storage of water in the channel and in inundated areas adjacent to the channel (Henderson, 1966). This storage is determined by the slope of the rating curve and the surface width of the channel and flood plain if over-bank flow occurs. Thus the routing of water is influenced by the flow resistance, through its effect on the rating curve, upstream of the point of observation.

What do the three types of data represent? It is clear from the processes determining the stage that the rating curve reflects the downstream control and downstream flow resistance. On the other hand the discharge time-series is determined by the upstream inflows of water and subsequent routing. Therefore the discharge reflects upstream storage processes which are partly influenced by upstream flow resistance and controls but also by storage of water outside the channel. The stage time-series represents a combination of upstream and downstream processes. Actual values of stage for a given discharge reflect the downstream processes and are equivalent to a rating curve. On the other hand the timing of fluctuations in stage are determined by the timing of fluctuations in discharge and thus depend on the upstream processes.

Flow resistance parameters calibrated using the different types of data can now be interpreted in terms of the above processes. If the stage-discharge relationship is used the calibrated parameter represents the average flow resistance between the point of observation and the controlling cross-sections further downstream plus any error in specifying the channel system downstream of the point of observation. When the discharge time-series is used the calibrated parameter represents the average upstream flow resistance plus any error in specifying the channel and any off-channel storage upstream of the point of observation.

When the stage time-series is used, the interpretation of the flow resistance parameter depends on the importance placed on the timing and magnitude of fluctuations in stage. If the timing is considered important then the flow resistance parameter is interpreted in the same way as when a discharge time-series is used. On the other hand if the magnitude of the fluctuations is important the interpretation is similar to the flow resistance obtained using a rating curve. When both characteristics are considered then both upstream and downstream processes contribute to the calibrated value of flow resistance.

Another factor to consider when calibrating a model is that there will be some errors in the model boundary conditions. These errors will be reflected in the simulated time-series of stage and discharge. Therefore the errors found in a comparison of observed and simulated time-series will reflect errors in both the model structure (ie. representation of the physical system) and parameterisation and in the upstream boundary condition and tributary inflows. Therefore the calibrated value of flow resistance will reflect errors in the upstream boundary condition and tributary inflows; although this influence may be relatively small, particularly if the errors in boundary conditions are random. The problem of errors in boundary conditions is significantly reduced if the rating curve is used, as one is then trying to calibrate the relationship between discharge and stage.

Of course, if the model is a correct representation of the system being modelled, the flow friction obtained by calibration should not depend on which data set is used in the calibration. The fact that this is not the case with this model of the Wimmera River is demonstrated and discussed in Chapter 10.

There is no ideal way to calibrate a hydrodynamic model simply using gauging station data. The different alternatives either emphasise the hydraulic controls and flow resistance processes immediately downstream of a gauge or emphasise storage processes upstream. It was decided to use rating curves to calibrate the model of the Wimmera River because of the errors inherent in the boundary conditions used for ungauged tributaries and because this method is most clearly related to flow resistance processes.

6.3.1.2 Flow Resistance Calibration.

Five gauging stations provide stage and discharge data and rating curves that can be used in calibrating the model; however one of these stations only provides data for high flows. These gauges only provide calibration data every 39 km on average. The data used to specify the river system and to specify lateral inflows is limited. It has been necessary to infill a significant number of cross-sections, lump anabranching channels together and represent the Huddlestons Weir diversion in a very simple manner. It was also necessary to estimate flows (and salinities) for a significant number of ungauged tributaries.

Given these approximations and that model predictions can only be tested at five locations it was decided that a simple parameterisation should be used. The model was calibrated with a single value of flow friction applying to all cross-sections and at all discharges with the exception of the water supply channels. For water supply channels, the flow resistance was set to 0.03. This value was chosen to reflect the relatively uniform nature of the water supply channels. Given that the water supply channels are only a small part of the model and that the interest is only in the flows in the channels, which are determined either from the discharge in the river or from a specified time-series, the flow resistance in this part of the model is unimportant.

Flow resistance was calibrated using rating curves and the following procedure. Firstly both the rating curve for each station and the predictions of stage and flow were plotted. This allowed a visual assessment of the goodness of fit. Secondly the predicted flow was used in conjunction with the rating curve to calculate an observed stage. This observed stage is not actually the stage recorded by the river gauge at that time but rather the stage that would be recorded by the river gauge given the predicted flow rate. This observed stage was then compared with the stage predicted by the model and a number of goodness of fit parameters were calculated.

Figure 6.8 shows the predicted and observed rating curves for the five gauging stations on the Wimmera River for the calibration period 15/6/89 - 10/10/89 with $n = 0.06$. This calibration period was chosen because a significant number of flow events occurred during this period and because 1989 was a relatively normal year for flows in the Wimmera River. All the rating curves and the simulated depths are referred to the cease to flow elevation except for the rating curve in Figure 6.8c. Since this gauging station is only rated for high flows, it was not possible to relate the rating curve to the cease to flow level. Its vertical position is therefore uncertain.

It can be seen from the various plots in Figure 6.8 that the model over-predicts the water level at some stations and under predicts it at others. Table 6.1 provides the mean error in predicted stage, μ_{Serr}, the coefficient of efficiency (Aitken, 1973) for the prediction of stage, $C_{eff.S}$, and the mean observed stage, μ_{Sob}, for different values of flow resistance for gauging stations at Glynwylln, Glenorchy, Horsham and Upstream of Dimboola. The stage-discharge relationship is specified at Lochiel and the Drung Drung is not rated for all flows so these gauging stations are excluded. These statistics were calculated using the observed stage, S_{obs}, which was calculated from the simulated discharge and the rating curve for each station and the simulated stage, S_{sim}. The above statistics were calculated using the following equations.

$$\mu_{Serr} = \frac{1}{n} \cdot \sum_{i=1}^{n} (S_{sim} - S_{obs}) \tag{6.5}$$

$$\mu_{Sob} = \frac{1}{n} \cdot \sum_{i=1}^{n} S_{obs} \tag{6.6}$$

Flow Resist.	0.05			0.06			0.07		
Gauge	μ_{Serr}	μ_{Sob}	C_{eff}	μ_{Serr}	μ_{Sob}	C_{eff}	μ_{Serr}	μ_{Sob}	C_{eff}
Glynwylln	0.03	0.86	0.98	0.16	0.86	0.88	0.28	0.86	0.69
Glenorchy	-0.12	0.74	0.88	-0.07	0.74	0.95	-0.02	0.74	0.98
Horsham	-0.22	0.90	0.86	-0.16	0.90	0.93	-0.09	0.90	0.96
U/S Dimboola	-0.03	1.31	0.99	0.05	1.32	0.98	0.14	1.33	0.94

Table 6.1: Model performance in predicting rating curves for different stations on the Wimmera River and for different flow resistances.

$$C_{eff} = \frac{(SSM - SSR)}{SSM} \qquad (6.7)$$

In Equations 6.5, 6.6 and 6.7 n is the number of observations, SSM is the sum of squared deviations from the mean observed stage and SSR is the sum of squared errors.

As can be seen from Table 6.1 and Figure 6.8, values of $n < 0.06$ are appropriate for the gauging stations at Glynwylln and Upstream of Dimboola and $n > 0.06$ are appropriate at Glenorchy and Horsham. Comparing the slope of the rating curve to that for the predicted stage-discharge relationship for Drung Drung (Figure 6.8c) indicates that a value of $n < 0.06$ is appropriate for this station. There is therefore no trend in appropriate values for n with distance downstream.

Some of the apparent variation in flow resistance along the river may be related to the fact that a number of the cross-sections immediately downstream of the gauging stations have been stochastically generated. Predictions of stage at a given locality are sensitive to the particular cross-sections used at and downstream of that locality (Chapter 5). This means that errors observed at a particular gauging station may be due to incorrectly specified cross-sections rather than to a change in the average flow resistance properties of the channel between gauging stations. The available data does not allow these two sources of error to be distinguished. Due to this factor and the fact that there was not a general trend in the flow resistance along the river channel a constant value of $n = 0.06$ was used.

6.3.2 ROUTING PERFORMANCE.

When a value of $n = 0.06$ is used for the flow resistance parameter the celerity of a simulated flood wave is significantly higher than the observed wave speed (Figure 6.9). There are a number of possible explanations for this. The first is that the value of n used is too small and thus the estimated storage is too small

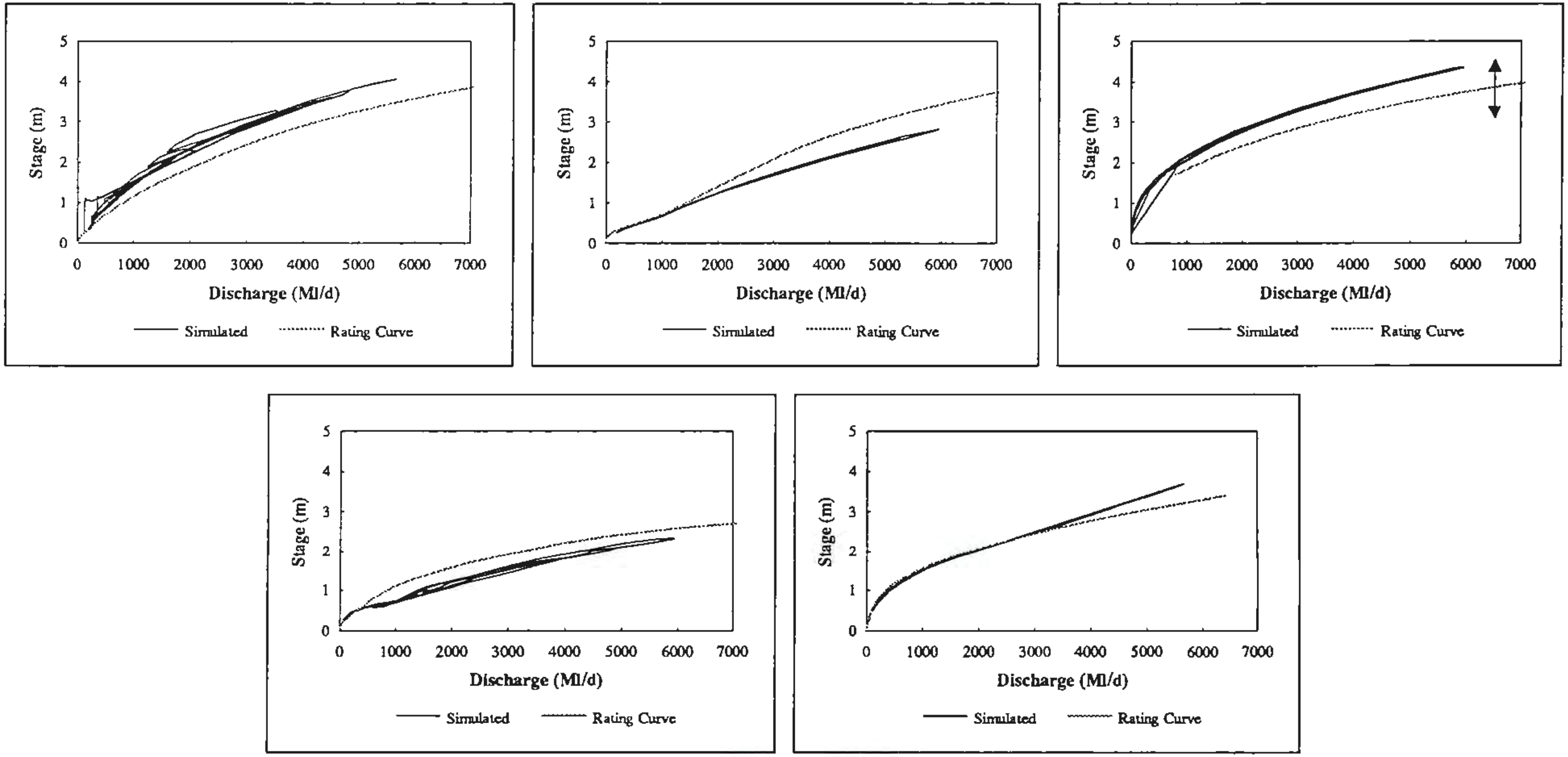


Figure 6.8: Simulated stage discharge relationships and rating curves for the Wimmera River a t(a) Glynwylln, (b) Glenorchy, (c) Drung Drung, (d) Horsham and (e) Upstream of Dimboola. Simulations are for the period 15/6/89 to 10/10/89 with n = 0.06.

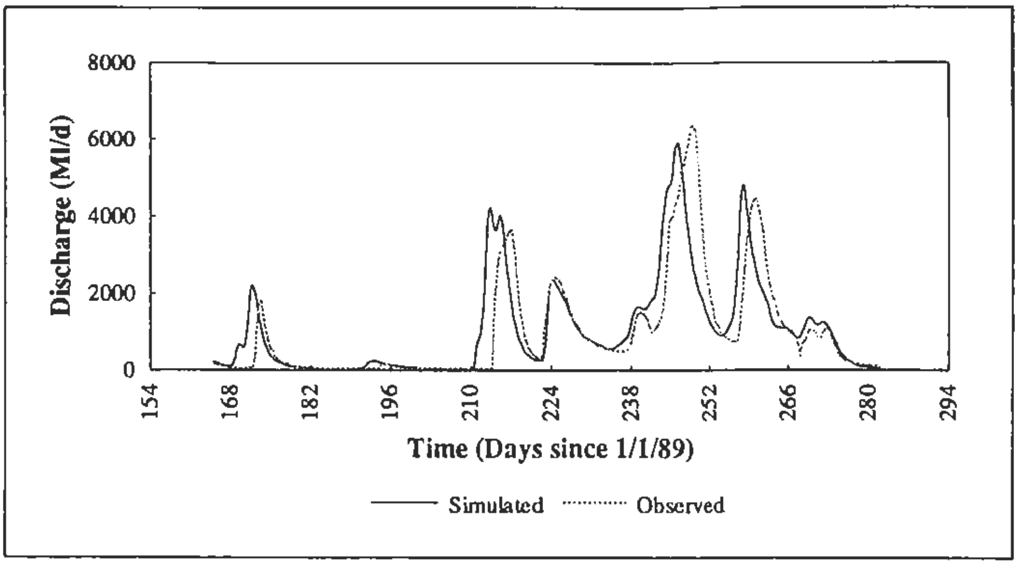


Figure 6.9: Simulated and observed hydrographs for the Wimmera River at Lochiel for the period 15/6/89 to 10/10/89 with n = 0.06.

and the wave celerity too large. It is also possible that the cross-sections used in the model are not representative.

If the width of the infilled cross-sections were biased such that they were too narrow; the predicted wave speed would be too high. Equation 6.8 gives the celerity of a Kinematic Wave, c, in terms of the water surface width, B, and the slope of the rating curve $\frac{\partial Q}{\partial H}$ (Henderson, 1966).

$$c = \frac{1}{B}\frac{\partial Q}{\partial H} \tag{6.8}$$

Given the calibration approach used, the simulated rating curves and thus $\frac{\partial Q}{\partial H}$ are approximately correct; at least at each gauging station. However if the cross-sections used in the model were too narrow (B too small), c would be over-predicted. The cross-sections were infilled on the basis of a rigorous statistical analysis of the available cross-sections and an error of approximately 50 % in the width is required to explain the difference in predicted and observed wave celerities (see §6.2.3). Furthermore, errors in the wave celerity persist between Horsham (415200) and Upstream of Dimboola (415256) where only 36% of cross-sections were infilled. This explanation of the error in wave speed is therefore considered unlikely.

Another possibility is that the conveyance through wetland sections has been overestimated by the effective channel used to model these sections. It is likely that the available cross-sections relating to these areas are representative of the largest channels flowing through the areas. Multiplying the cross-sectional area by the number of channels estimated to flow through the area may then lead to an overestimate of conveyance. The stage and possibly the storage in and upstream of these areas would then be underestimated and water would be routed through the areas too quickly.

Alternatively there may be additional sources of storage that have not been taken into accounted. Two possible sources of additional storage exist. Firstly, there are backwater areas along the river which are associated with relict stream channels. In some areas, especially areas where a number of anabranches exist, this could be a significant source of storage. During field visits inundation of backwater areas was observed in both wetlands and other areas for discharges of between 1 000 Ml/d and 2 000 Ml/d. Secondly, since in-bank flows are of greatest interest, no attempt has been made to build in the effects of the flood plain. This would have a significant affect on the simulation of events for which significant flood plain inundation occurs. Minor flood warnings for the Wimmera River downstream of Glenorchy are issued by the Bureau of Meteorology when the stage at the Glenorchy gauge exceeds 4.0 m (K. Sondberg, B. Met., Per. Com.) which is equivalent to a discharge of approximately 6 600 Ml/d. At this discharge low lying areas adjacent to the main stream are inundated. The model could not be considered reliable for flows exceeding this value since the flood plain has not been modelled. It is likely that a combination of backwater areas and inundation of low lying flood plains, especially in wetland or anabranching areas, is responsible for the errors in the simulated flood wave celerity.

Since little information was available on additional storage along different parts of the river, the error in wave celerity was corrected by using a calibration approach. A new parameter, ω, was introduced into the model which simply increased the storage width, B, to an effective storage width, B', using Equation 6.9.

$$B' = \omega * B \tag{6.9}$$

In practice this was achieved by calculating an additional storage area for each elevation specified in the table of cross-sectional parameters and including this value as a flooded area. Equation 6.10 was used to calculated the additional storage.

$$A_f = 0.5 * B_z * (X_{n+1} - X_{n-1}) * (\omega - 1) \tag{6.10}$$

Gauging Station	$\omega = 1.4$	$\omega = 1.5$	$\omega = 1.6$
Glenorchy	0.91	0.91	0.90
Horsham	0.82	0.85	0.87
U/S Dimboola	0.85	0.87	0.88
Lochiel	0.86	0.88	0.88

Table 6.2: Coefficients of Efficiency for the prediction of 6 hourly instantaneous discharge for n = 0.06 and different values of ω for four gauges for the calibration period.

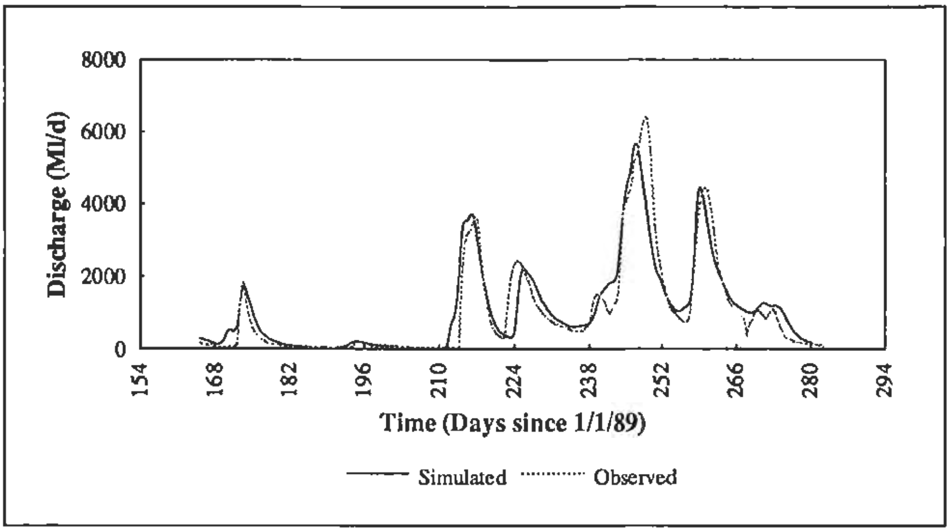
In Equation 6.10 A_f is the flooded area, B_Z is the storage width at elevation Z, X_{n-1} and X_{n+1} are the upstream and downstream cross-section chainages respectively and ω is the parameter used to increase the storage width.

Figure 6.10 shows the predicted and observed hydrographs for the Lochiel gauging station and Table 6.2 gives the coefficient of efficiency for the prediction of discharge for the Wimmera River at Glenorchy, Horsham, Upstream of Dimboola and Lochiel. Glynwylln has been excluded because the discharge is specified there.

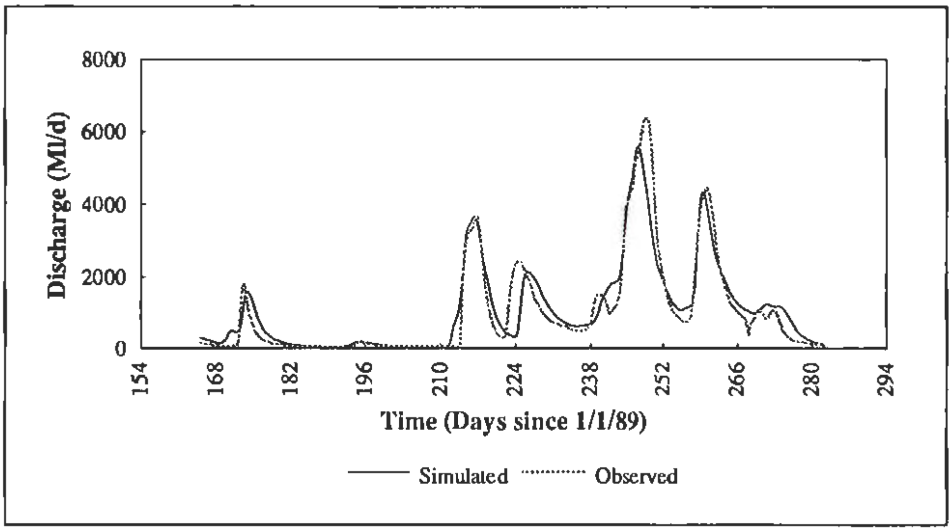
The values of the coefficient of efficiency supplied in Table 6.2 are insensitive to V. This is partly due to the significant errors in the discharge predictions resulting from the incorrect specification of inflows from ungauged tributaries and outflows to the WMSDS and Yarriambiack Creek. Errors in the prediction of mean discharge at the various gauging stations are evidence of such erroneous specification. Since the coefficient of efficiency was so insensitive to ω, a value of 1.5 was chosen visually using the timing of the rise of the hydrographs as the main consideration (Figure 6.10). It is noted that the errors in the predicted hydrographs are due to errors in the magnitude and timing of various inflows as well as due to errors in the wave celerity and that it is difficult to separate these various sources of error. There is therefore a significant amount of uncertainty associated with the value of ω chosen.

6.3.2.1 Interpretation of ω.

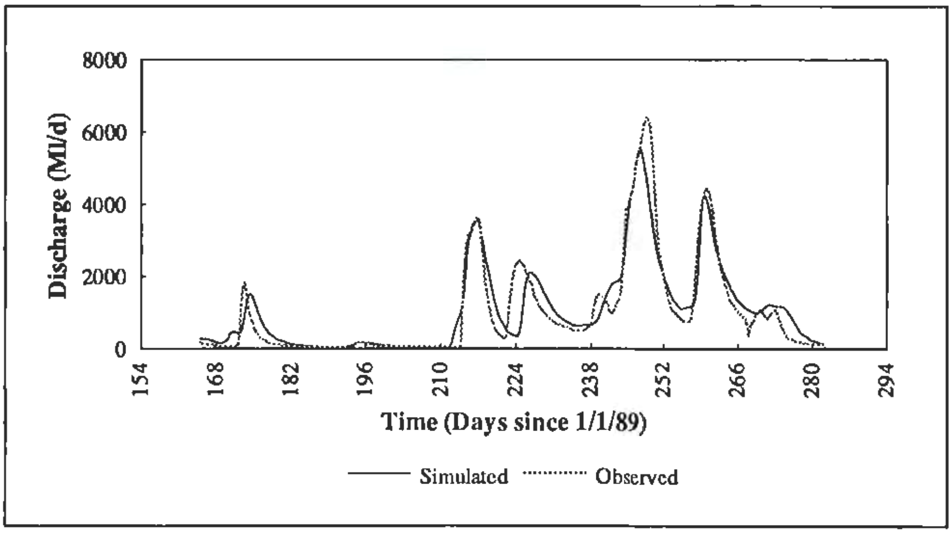
How can ω be interpreted? The effect of ω in the model is to increase the storage of water during a flow event. It can be interpreted as representing a correction for a number of sources of error the most significant of which is likely to be the combined effect on storage of backwater areas in relict channels and inundation of low lying areas along the stream. ω modifies both the flood wave celerity, and the attenuation of the flood wave. The flood wave celerity can be approximated



(a)



(b)



(c)

Figure 6.10: Simulated and observed hydrographs for the Wimmera River at Lochiel for the period 15/6/89 to 10/10/89 with n = 0.06 and (a) ω = 1.4, (b) ω = 1.5 and (c) ω = 1.6.

by the kinematic wave speed provided the wave is slowly rising (Henderson, 1966). The kinematic wave speed is given by Equation 6.8 and the modified wave speed, c', for a given discharge is obtained by combing Equations 6.8 and 6.9.

$$c' = \frac{c}{\omega} \tag{6.11}$$

The spatial rate of subsidence of a flood wave also depends on the wave celerity as in Equation 6.12 (Henderson, 1966) which implies that the subsidence will increase as W^2 increases (Equation 6.13).

$$\frac{\partial y}{\partial x} \alpha \frac{1}{c^2} \tag{6.12}$$

$$\left(\frac{\partial y}{\partial x}\right)' = \omega^2 \frac{\partial y}{\partial x} \tag{6.13}$$

It is probable that a significant amount of the storage associated with these features of the river is concentrated in the sections where anabranches are prevalent. Such sections occur frequently along the part of the river included in the model. While it is possible to identify areas where the stream is typically confined to a single channel or is typically anabranching; it was considered that trying to treat these two typical sections differently was not practical because data were only available at gauging stations located significantly further apart than the typical length of the individual sections.

6.4. TESTING THE HYDRODYNAMIC MODEL.

6.4.1 ROUTING PERFORMANCE.

Figures 6.11, 6.12, 6.13 and 6.14 show hydrographs for periods when flow events occurred in the river downstream of Horsham during 1990, 1991, 1992 and 1993 respectively. Table 6.3 provides the mean error in predicted discharge, μ_{Qerr}, and the coefficient of efficiency for the prediction of six hourly instantaneous discharge for these years. In 1990 only one significant flow event occurred downstream of Huddlestons Weir. This was a flood which was of a larger magnitude than the model is intended to simulate. While it is not intended that the model be applicable to such large events, it is noted that the poor performance of the model is largely due to the significant transmission loss which occurred as the event moved downstream. This event does indicate that significant transmission losses can occur under some circumstances. These losses have not

	Glenorchy 415201			Horsham 415200			Upstream Dimboola 415256			Lochiel 415246		
Year	μ_{Qerr} (Ml/d)	μ_{Qerr} (%)	C_{eff}	μ_{Qerr} (Ml/d)	μ_{Qerr} (%)	E_Q	μ_{Qerr} (Ml/d)	μ_{Qerr} (%)	C_{eff}	μ_{Qerr} (Ml/d)	μ_{Qerr} (%)	C_{eff}
1990	-2.8	-1.4	0.93	4.4	13.0	-0.45	23.7	78.3	-0.07	27.3	102.9	-0.09
1991	-22.2	-7.8	0.91	-36.0	-19.1	0.72	-55.0	-22.0	0.83	-67.4[a]	-19.4[a]	0.83[a]
1992	-13.7	-2.1	0.91	-197	-21.3	0.65	-30.8[b]	-4.8[b]	0.73[b]	-213	-20.4	0.72
1993	-39.5[a,b]	-7.4[a,b]	0.85[a,b]	-45.7	-16.1	0.58	-23.0[b]	-.7.4[b]	0.76[b]	13.4[a,b]	6.5[a,b]	0.78[a,b]

Table 6.3: Mean annual errors in simulated discharge and coefficient of efficiency for the prediction of instantaneous flow for Wimmera River Gauging Stations. a - some low flows missing. b - some high flows missing.

occurred during the first events of the other years simulated. This model of the Wimmera River is not capable of modelling such losses.

During 1991 a number of events passed downstream of Huddlestons Weir. The model significantly underestimated the peak discharges and total discharges for the first four events (day 185, day 190, day 225 and day 235 at Lochiel) which indicates that errors in boundary conditions rather than errors in the routing of water down the stream are the most significant source of simulation error. It also indicates the problems inherent in trying to calibrate a model when there is significant uncertainty in the boundary conditions. The two largest events were simulated adequately. It is noted that the four smaller events earlier in this period would have been significant from the point of view of mixing of saline pools.

In 1992 a variety of events passed downstream of Huddlestons Weir. Four of these events had peak discharges at Glenorchy which exceeded the minor flood level and are thus larger than the model was developed to simulate. It is noted that the total discharge in several of the events was underestimated which indicates that some significant inflows downstream of Glenorchy have been underestimated. There is also a tendency for the model to underestimate the travel time for the largest events which is due to the exclusion of the flood plain from the model. The model performed well for smaller events in 1992; especially when compared to 1991.

During 1993 only a small number of events passed downstream of Huddlestons Weir. The three largest of these events had peak discharges in excess of the minor flood level at Glenorchy. The model performed relatively well during 1993 except that simulated total and peak discharges were inaccurate for some events.

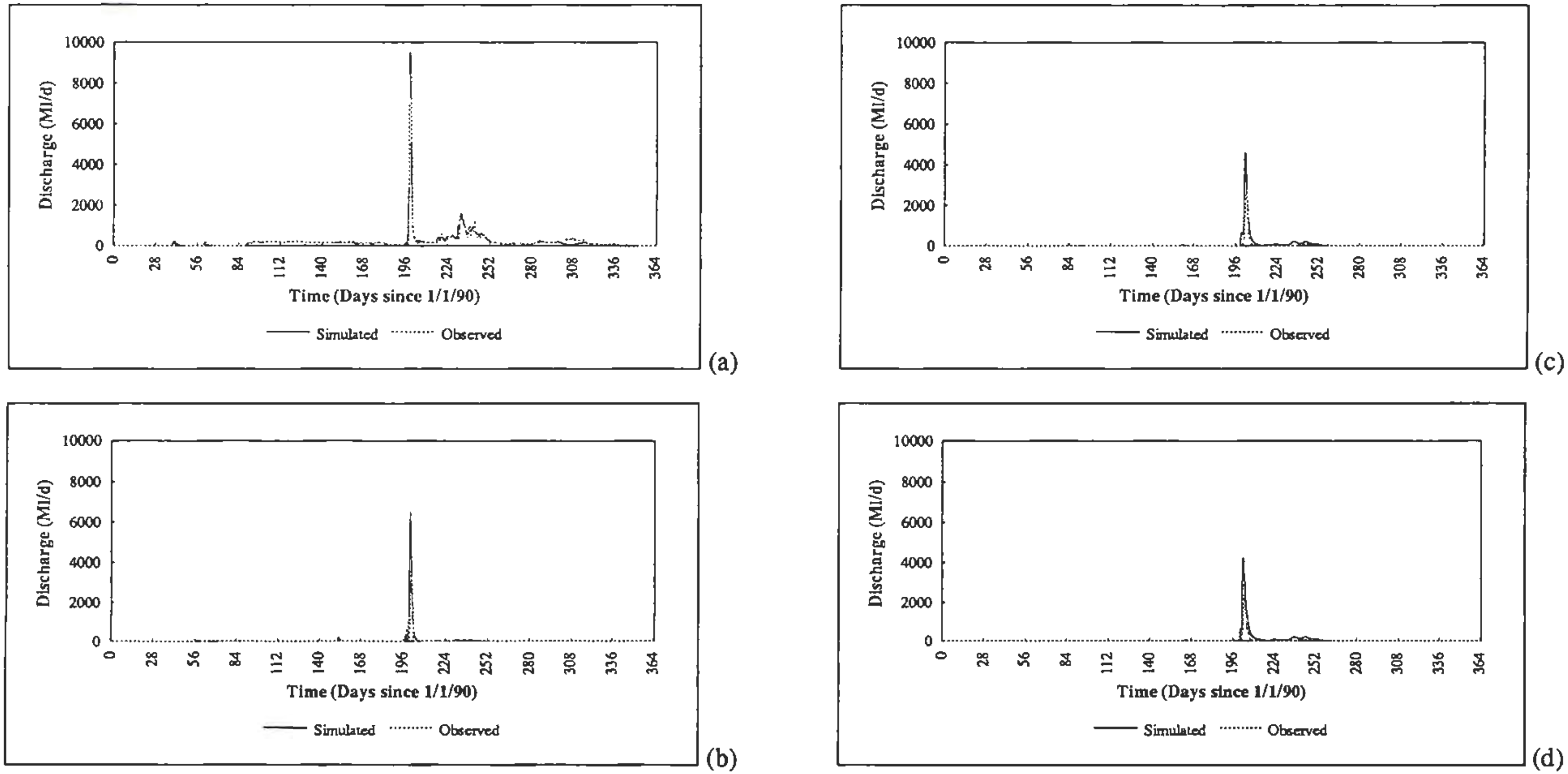


Figure 6.11: Simulated and observed hydrographs for the Wimmera River at (a) Glenorchy, (b) Horsham, (c) Upstream of Dimboola and (d) Lochiel for 1990. Simulations conducted with $n = 0.06$ and $\omega = 1.5$.

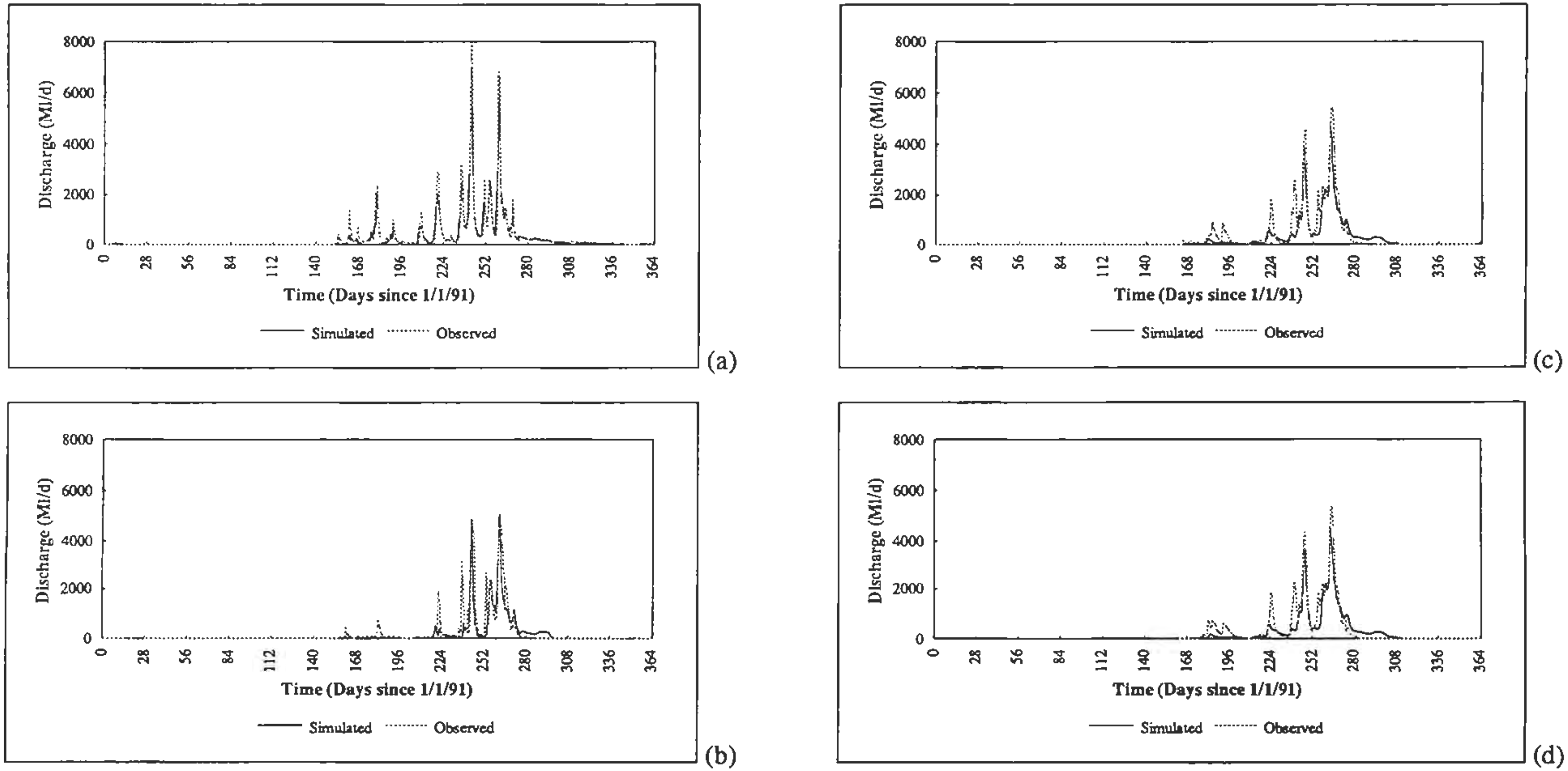


Figure 6.12: Simulated and observed hydrographs for the Wimmera River at (a) Glenorchy, (b) Horsham, (c) Upstream of Dimboola and (d) Lochiel for 1991. Simulations conducted with n = 0.06 and $\omega = 1.5$.

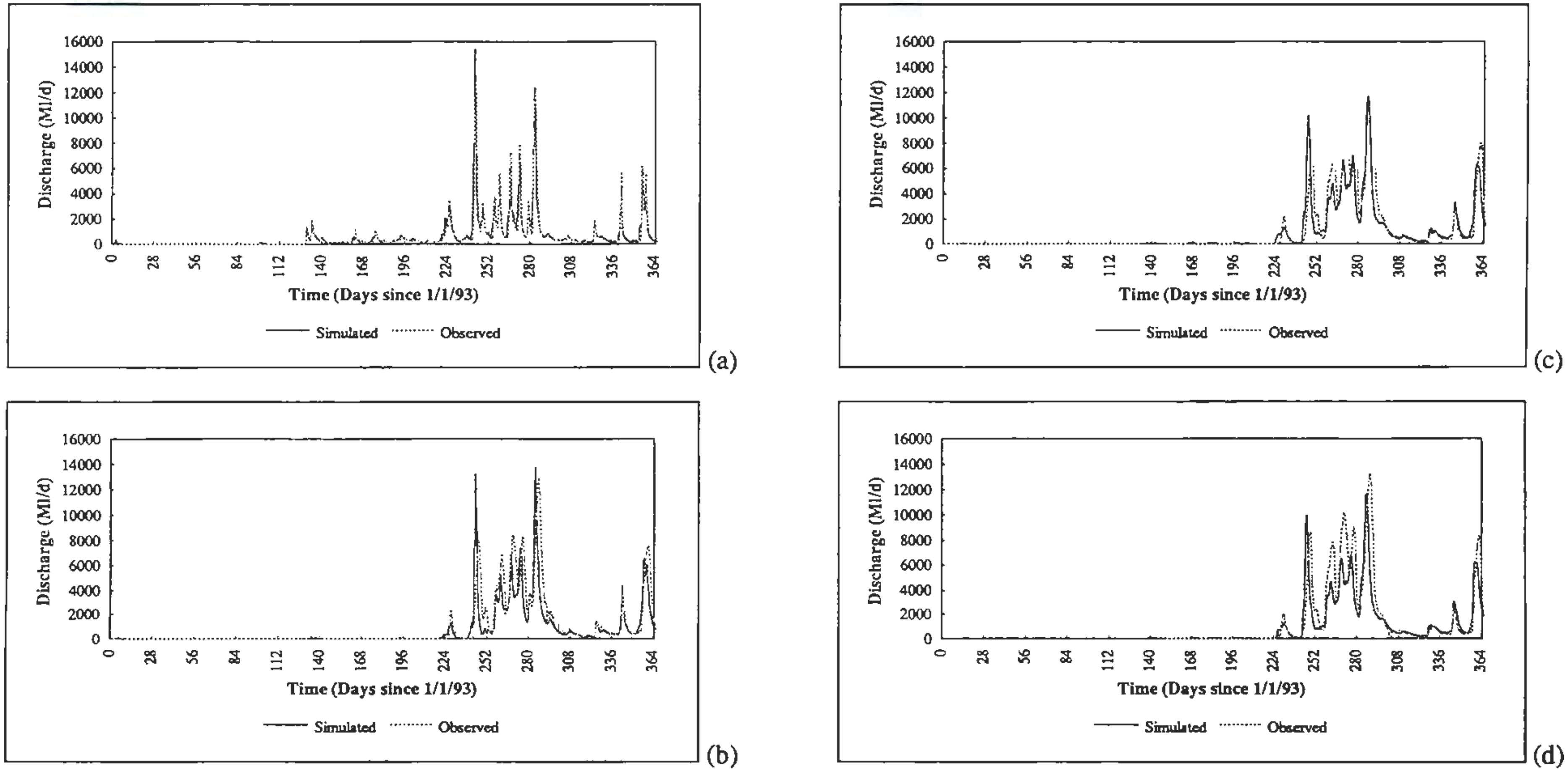


Figure 6.13: Simulated and observed hydrographs for the Wimmera River at (a) Glenorchy, (b) Horsham, (c) Upstream of Dimboola and (d) Lochiel for 1992. Simulations conducted with n = 0.06 and $\omega = 1.5$.

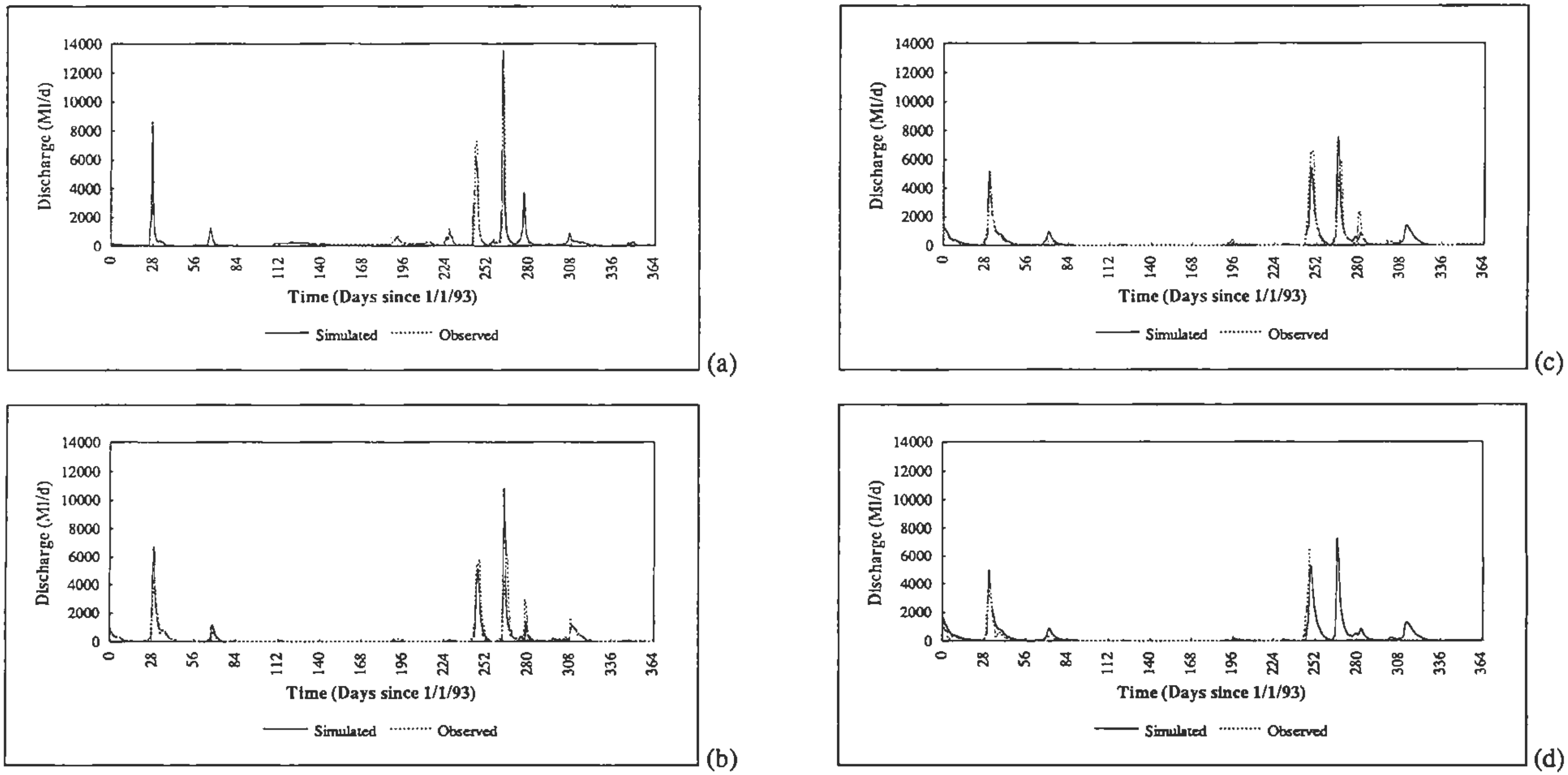


Figure 6.14: Simulated and observed hydrographs for the Wimmera River at (a) Glenorchy, (b) Horsham, (c) Upstream of Dimboola and (d) Lochiel for 1993. Simulations conducted with n = 0.06 and ω = 1.5.

The following comments can be made about the model. Firstly, the discharge simulations at Glenorchy are good. The quality of predictions at Horsham, Upstream of Dimboola and Lochiel tends to be similar for a given event, but is variable between events. Errors in the simulated total volume for individual events indicate that errors in estimating tributary inflows are responsible for poorer quality prediction at these stations. Assuming that this is the case, and noting the most tributary inflows occur upstream of Horsham, it is not surprising the similar errors occur at each of the three downstream gauging stations for a given event. There is a general tendency for the peak discharges and total discharges for events to be underestimated at the downstream gauges; however on some occasions the opposite is true. A significant amount of the error must be related to poorly specified boundary conditions because the total volume passing a gauge during an event is in substantial error. This is a direct consequence of the number and significance of ungauged tributaries.

Secondly, there are timing errors which occur in the predictions. These arise from two sources. The first is the crude way in which additional storage effects have been included and the total lack of inclusion of the significant flood plain inundation that occurs during larger floods. The second is errors in the timing of tributary inflows which have been estimated for ungauged tributaries.

6.4.2 HUDDLESTONS WEIR DIVERSIONS.

Figure 6.15 show the observed and simulated flows in the Wimmera Inlet Channel for 1990, 1991, 1992 and 1993. The observed flow in the Wimmera Inlet Channel was calculated by adding the flow in the Wimmera Inlet Channel at McKenzies Drop to the flow in the Rocklands Channel and subtracting the flow in the Lubeck Loop Channel (Figure 6.16). This represents the flow diverted from the Wimmera River, Mt William Creek and other tributaries flowing into the Wimmera Inlet Channel. Table 6.4 provides the mean observed flow, μ_{Qobs}, the mean error, μ_{Qerr}, and the coefficient of efficiency for the prediction of mean daily flow.

The model generally predicts diversions from the Wimmera River to the Wimmera Inlet Channel well, especially during 1990, 1991 and 1992. There are however some periods when the model consistently under-predicts the discharge. On some occasions (such as days 253 to 290, 1991) this is due to the assumption that no diversion was occurring. However in reality there may have been a supplementary diversion from the Wimmera River to Pine and Taylors Lakes or a

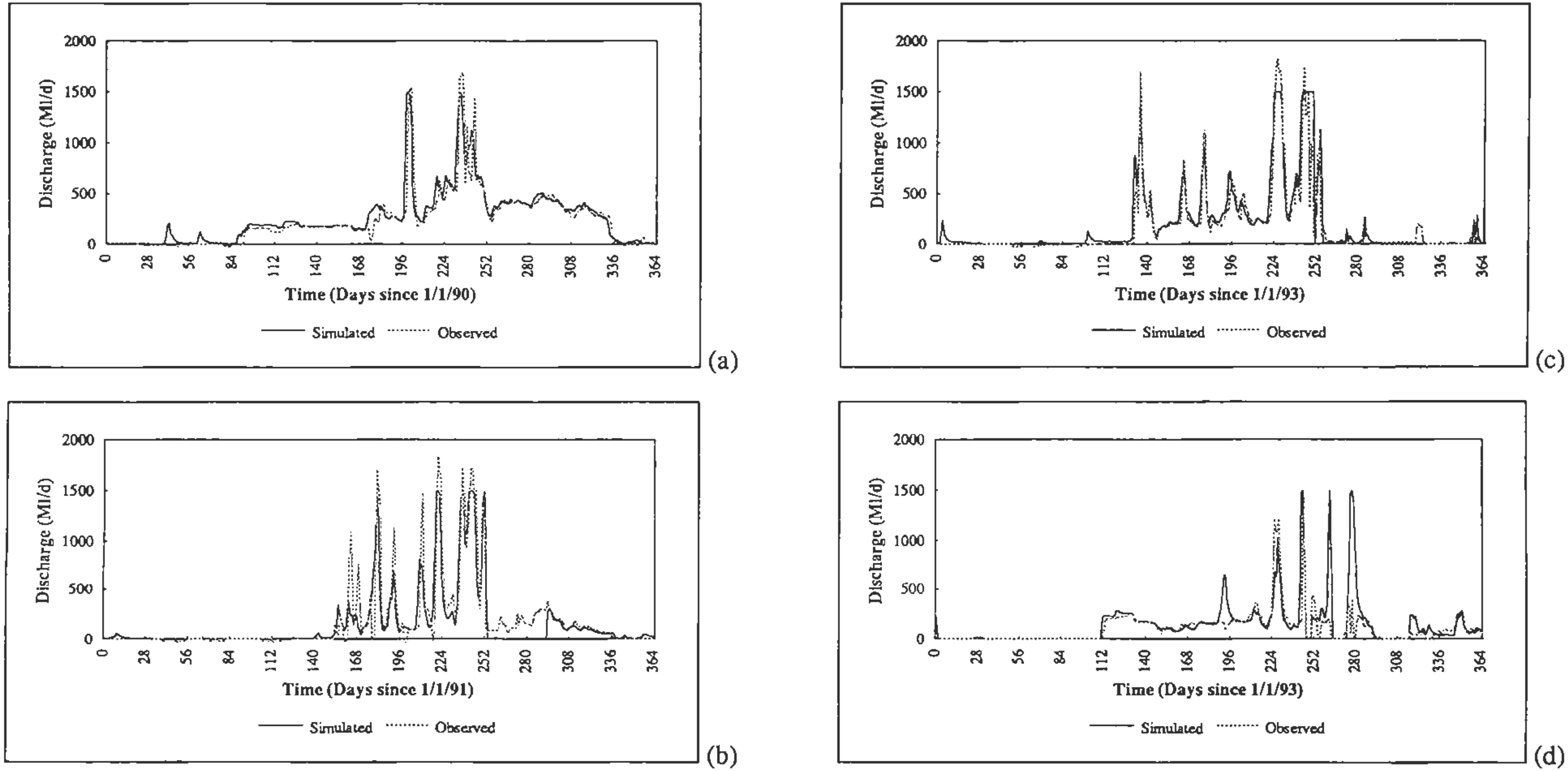


Figure 6.15: Simulated and observed diversions from the Wimmera River to the Wimmera Inlet Channel for (a) 1990, (b) 1991, (c) 1992 and (d) 1993. Simulations conducted with n = 0.06 and $\omega = 1.5$.

Year	μ_{Qobs} (Ml/d)	μ_{Qerr} (Ml/d)	μ_{Qerr} (%)	C_{eff}
1990	237	22.7	9.6	0.82
1991	178	-24.7	-13.9	0.76
1992	167	30.5	18.3	0.81
1993	110	32.6	29.8	-0.33

Table 6.4: Mean annual errors and coefficient of efficiency for the prediction of mean daily discharge in the Wimmera Inlet Channel.

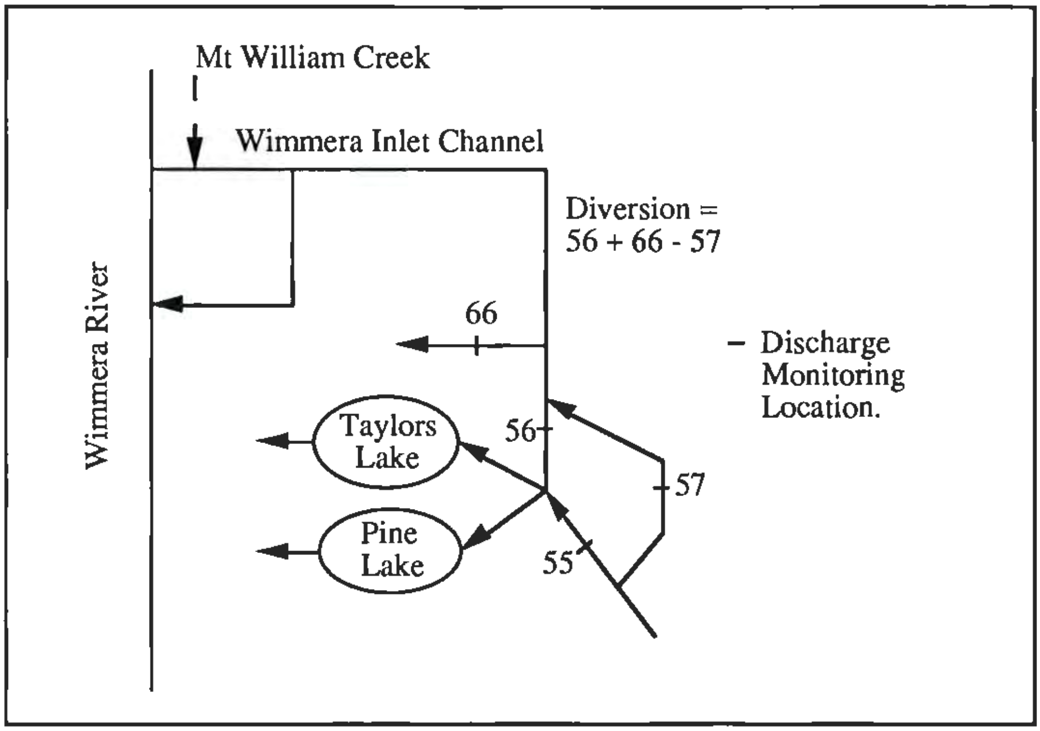


Figure 6.16: Monitoring sites used to calculate the diversion from the Wimmera River to the Wimmera Inlet Channel.

diversion from the river to the Rocklands Channel in which case only part of the total river flow may have been diverted.

During 1993 diversions from the river were significantly over-estimated during three events and these errors are reflected in the negative coefficient of efficiency (Table 6.4). On these occasions the diversion was modelled as though all possible diversions were being made when in reality the diversion was being limited to a smaller rate. There are also many periods where the pattern of diversions has been modelled well but where there are also relatively small errors. These errors may result from incorrectly simulated river discharge upstream of the diversion, incorrectly estimated flows in Mt William Creek or errors in the observed diversion itself. While the diversion from the Wimmera River to the Wimmera

Inlet Channel is modelled accurately on many occasions, there are some occasions where the assumption that the diversion is either at the maximum possible rate or zero is inadequate.

6.5 TRANSPORT DISPERSION SIMULATIONS.

MIKE 11 simulates the transport of solutes using the Advection-Dispersion equation. Results from the hydrodynamic simulations are used to calculate advective velocities, cross-sectional areas and storage volumes. Application of MIKE 11 to the simulation of salt transport in the Wimmera River is described below.

6.5.1 DISPERSION COEFFICIENT.

The dispersion coefficient, D, is the only parameter in the advection-dispersion equation that needs to be set. A sensitivity test for the period 1/8/92 - 31/8/92 was performed using dispersion coefficients of 50 and 500. Figure 17 shows a comparison of these two simulations for the Wimmera River at Upstream of Dimboola. Given the order of magnitude change in D, there is little difference between the two. Therefore the dispersion coefficient was simply specified using a relationship (Equation 6.14) given by Fischer et al. (1979).

$$D = \frac{0.011 U^2 B^2}{d U_*} \tag{6.14}$$

In Equation 6.14 U is the mean velocity, B is the water surface width, d is the depth and U_* is the shear velocity. Using Manning's Equation and $U_* = \sqrt{gRS}$ it is possible to obtain:

$$U_* = \sqrt{g}\, R^{-1/6}\, U\, n\ . \tag{6.15}$$

Approximating R with d and substituting Equation 6.15 into Equation 6.14 leads to:

$$D = \frac{0.011}{\sqrt{g}} \frac{U B^2}{d^{5/6}\, n} = k_D\, U\ . \tag{6.16}$$

If B varies between 20 and 80, d varies between 1 and 5 and n = 0.06 then k_D will vary between 6 and 370. Since the model of the Wimmera River is insensitive to D, Equation 6.16 was used with k_D set to 100 and D limited such that $50 < D < 300$. No calibration of the Advection Dispersion model was attempted. Dispersion coefficients were set to zero at weirs to prevent dispersive transport at these locations during periods of zero flow.

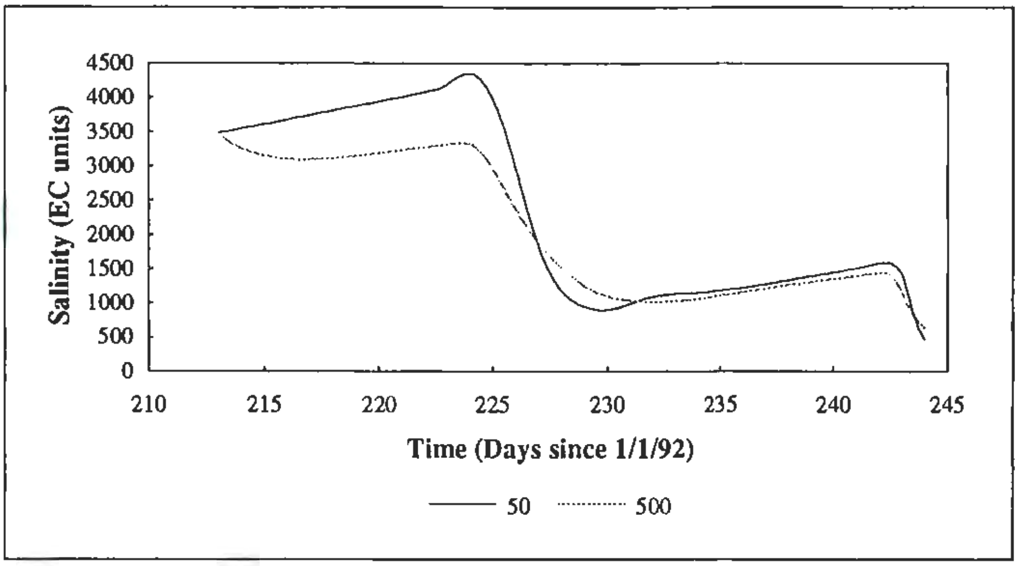


Figure 6.17: Salinity predictions for the Wimmera River at Upstream of Dimboola with dispersion coefficient = 50 and dispersion coefficient = 500.

6.5.2 ADVECTIVE COURANT NUMBER.

In order to ensure an accurate solution to the Advection Dispersion Equation the time-step in solute transport calculations should be limited so that the Advective Courant Number, $C_r = U.\Delta t/\Delta x$, is less than 1 where U is the mean velocity, Δt is the time-step and Δx is the space-step (DHI, 1992b). However this would require time-steps of less than 1 minute in the Wimmera model. Since simulations performed with time-steps of 1 minute and 7.5 minutes were almost identical the longer time-step was used and the above Courant Number condition was ignored. While some error is introduced into the simulations in this manner it is much smaller than errors arising from poorly specified boundary conditions.

6.5.3 INCLUSION OF GROUNDWATER INTERACTION.

While groundwater interaction was not included in the hydrodynamic simulations it has been included in the solute transport simulations because groundwater is an important source of salt. Two sources of information have been used to determine groundwater inflows. Firstly, for the upper river, the analysis in Chapter 4 provides inflow rates for low flow periods. Groundwater discharge was assumed to be constant for the river upstream of Glenorchy and salt was added at 4 locations along this reach. While some error can be expected during high flow periods using this approach this is not large due to the insignificance of groundwater as a source of salt during high flows. Groundwater inflows for the reach of river between Glenorchy and Roseneath were assumed to be zero (see Chapter 4).

Secondly, groundwater information is available from an unpublished study conducted by the Centre for Land Protection Research (D. Strudwick, DCE, Per. Com.). This study provides groundwater flows for a number of locations between Roseneath and Lochiel which are based on measured groundwater and surface water elevations and flow net analyses. The quality of these estimates is unclear; however since an investigation of interaction between the groundwater and the river over this reach was outside the scope of this study, these estimates were used as supplied. Groundwater inflows over this reach of the Wimmera River were again treated as steady but varied in space. Groundwater inflows were specified approximately every 1000 m.

The interaction of the groundwater and the river obviously has a dynamic component; however this was not included in the model for two reasons. Firstly, there was little information on this dynamic component. Secondly, an empirical relationship between river stage and groundwater inflow or some form of linking between river stage and water table elevation would have been required in the model. Additional model modifications would have been required to include either of these options.

6.5.4 TESTING THE SOLUTE TRANSPORT MODEL.

Figures 6.18 and 6.19 show the observed and simulated salinities for periods when continuous salinity data are available at several gauging stations. Observed stream discharge is also shown so that flow events can be identified. Tables 6.5 and 6.6 provide the mean errors, prediction efficiencies and mean observed salinities for each station. Unfortunately there are significant periods where salinity data are not available due to malfunction of the RWC salinity monitoring equipment. The salinity boundary condition at Glynwylln was obtained using a solute rating curve (see Chapter 4).

Salinity predictions are most accurate at Upstream of Glenorchy (415258) and become less accurate at Horsham (415200) and Upstream of Dimboola (415256) (Figures 6.18 and 6.19). The largest errors in predicted salinity occur during low flow periods at Horsham and at Upstream of Dimboola. During high flow periods the error in predicted salinity rarely exceeds and is generally significantly less than 50% of the observed salinity. The pattern of salinity fluctuations at high flows is well modelled, especially at Upstream of Glenorchy. The association between salinity fluctuations and discharge fluctuations, which is high at

			Solute Rating Curve			Monitored Salinity		
Station	n	μ_{ECobs} (EC)	μ_{ECerr} (EC)	μ_{ECerr} (%)	E_{EC}	μ_{ECerr} (EC)	μ_{ECerr} (%)	E_{EC}
U/S of Glenorchy	581	1250	-90	-7.5	0.52	-200	-15.9	0.58
Horsham	791	1160	-320	-27.4	0.33	-360	-31.2	0.35
U/S of Dimboola	220	640	450	71.2	-85	450	70.4	-84

Table 6.5: Mean errors and coefficient of efficiency for the prediction of six hourly instantaneous salinity for three salinity monitoring stations on the Wimmera River for the period 17/6/92 to 31/12/92. A solute rating curve is used to estimate river salinity at Glynwylln for the first simulation and monitored salinities are used in the second simulation. The first simulation The number of observations and mean observed salinity are also included.

			Solute Rating Curve			Monitored Salinity		
Station	n	μ_{ECobs} (EC)	μ_{ECerr} (EC)	μ_{ECerr} (%)	E_{EC}	μ_{ECerr} (EC)	μ_{ECerr} (%)	E_{EC}
U/S of Glenorchy	682	2160	-50	-2.4	0.51	70	3.3	0.59
Horsham	1243	1080	-210	-19.5	0.26	-150	-13.8	0.54
U/S of Dimboola	1461	1360	2330	171	-56	2330	171	-58

Table 6.6: Mean errors and coefficient of efficiency for the prediction of six hourly instantaneous salinity for three salinity monitoring stations on the Wimmera River for 1993. A solute rating curve is used to estimate river salinity at Glynwylln for the first simulation and monitored salinities are used in the second simulation. The number of observations and mean observed salinity are also included.

Upstream of Glenorchy, decreases downstream. Given that the salinities of all inflows and the discharges for many of the inflows have been estimated, the model predicts salinity well during high flows. The similarity of salinities at Horsham and Upstream of Dimboola indicate that stream salinities in the lower Wimmera River are dominated by the upstream inflow during periods of high flow.

While the predicted salinity at Upstream of Glenorchy is relatively good during low flows, significant errors can occur during periods of low flow at Horsham and at Upstream of Dimboola. The Horsham gauging station is located approximately 4 km downstream of the Horsham weir pool. No groundwater inflows have been specified in the model between these two locations hence the simulated salinity at the Horsham gauging station does not increase dramatically during low flow periods. The model under-predicts the salinity at this site during low flow periods. One possible explanation for this behaviour is that there is

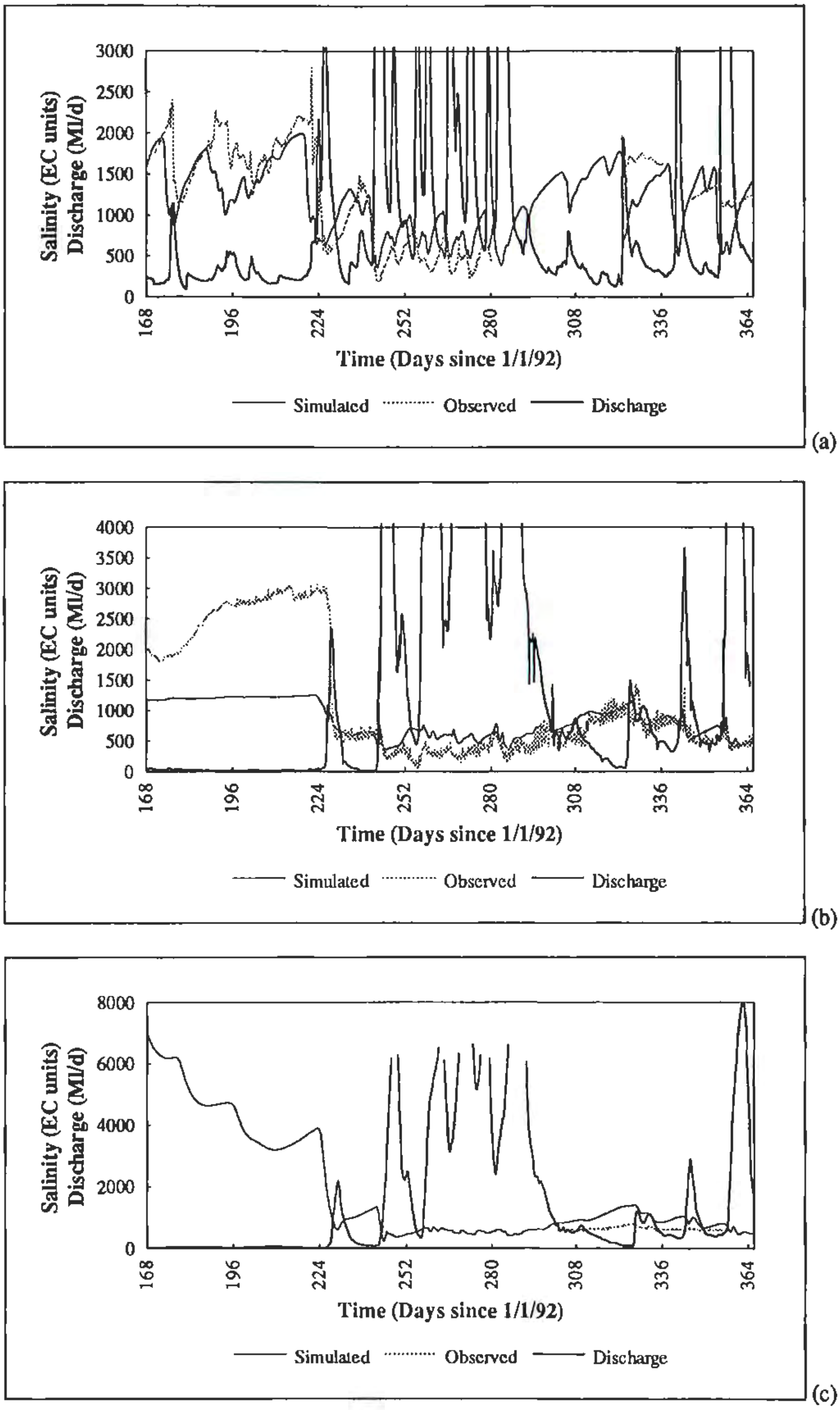


Figure 6.18: Simulated and observed salinities for the Wimmera River at (a) Upstream of Glenorchy, (b) Horsham and (c) Upstream of Dimboola for the period 17/6/92 to 31/12/92. Salinity boundary condition at Glynwylln specified using a solute rating curve.

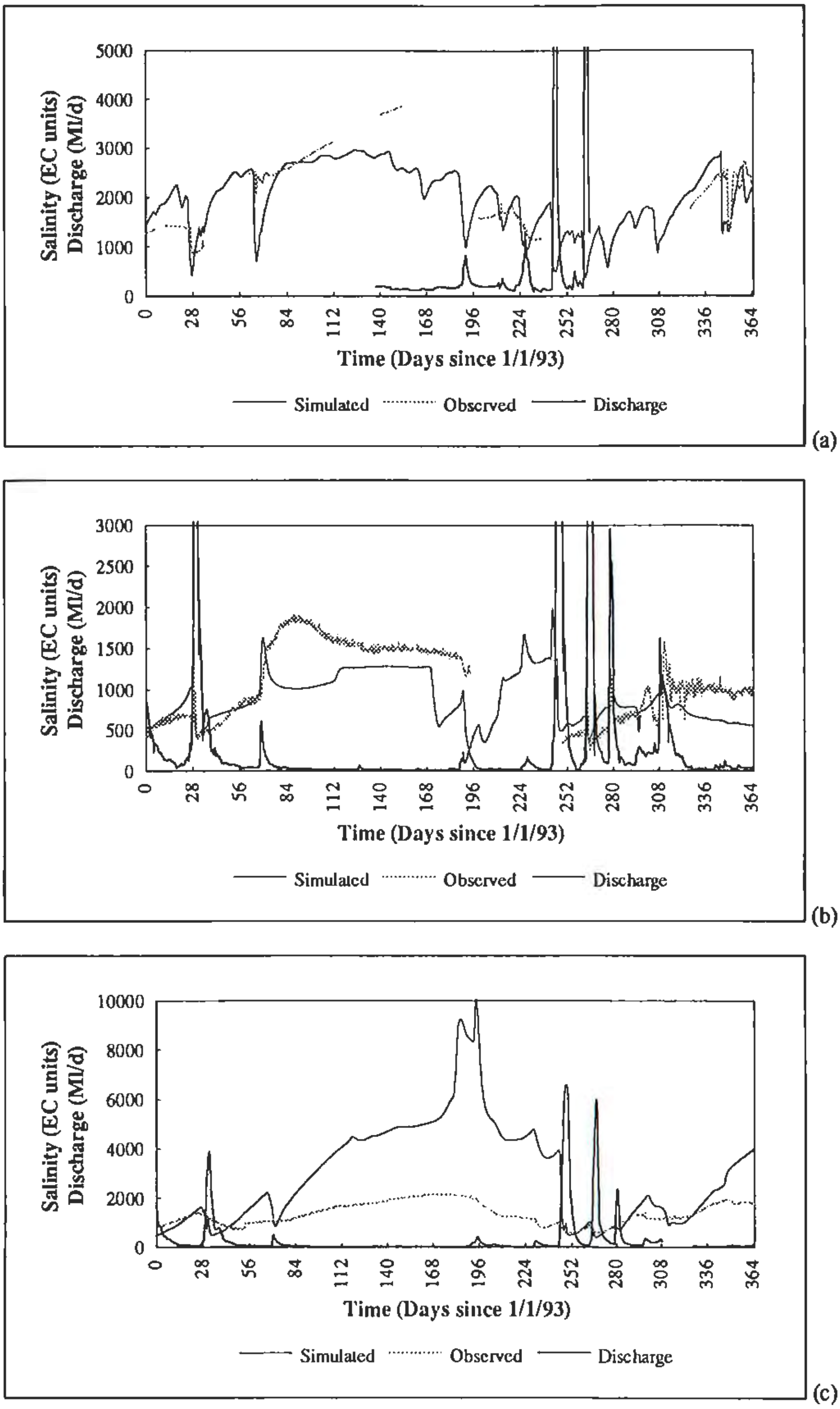


Figure 6.19: Simulated and observed salinities for the Wimmera River at (a) Upstream of Glenorchy, (b) Horsham and (c) Upstream of Dimboola for 1993. Salinity boundary condition at Glynwylln specified using a solute rating curve.

some groundwater entering the river between the Horsham Weir and the Horsham gauging station; however further investigations would be required to confirm this.

During low flow periods, simulated salinities gradually increase over time and with distance downstream between Horsham and Upstream of Dimboola due to the influence of groundwater inflows. The computational point representing the gauging station Upstream of Dimboola is located at one of the low flow weirs and the dispersion coefficient is zero there. This means that salinities at this location are representative of the salinity of water passing the gauge when the stream is flowing but that they are not representative of the general stream salinity during periods of zero flow. Several sources of error may contribute to the over-estimation of salinity in the lower river. Firstly errors in the simulated discharge and storage of water in the channel will lead to erroneous salinity predictions due to the effect on the dilution of the inflowing groundwater. Secondly errors in the groundwater discharge and thus the salt influx are likely to be present. Thirdly saline pools have been ignored. Inclusion of saline pools would be expected to lead to lower surface water salinities due to storage of salt in the saline lower layer.

Figures 6.20 and 6.21 the simulated and observed salt fluxes at Horsham and Upstream of Dimboola. While significant errors in salinity predictions occur at low flows, particularly at Upstream of Dimboola, the error in the salt flux during those periods is relatively small because the flow is so low. Figures 6.20 and 6.21 show that transport of salt in the Wimmera River is dominated by high flow events (see Figures 6.18 and 6.19 for discharges). Given that the salt influx specified in the model associated with groundwater inflows between Horsham and Lochiel is only 70 t/d it is insignificant compared to the salt flux at Horsham during high flow periods.

While absolute salinities predicted by the model may be in significant error, particularly during low flow periods, the model simulates the behaviour of stream salinity, including features such as the reduction in salinity levels during flow events, well given the estimation of inflow rate and salinities. Sources of error in the solute transport model include errors in the hydrodynamic simulations, errors in the specification of the magnitude and salinity of surface water inflows, errors in the magnitude and salinity of groundwater inflows, errors in the magnitude of outflows and errors in the process description. Given the uncertainty in many of the boundary conditions, the model cannot be expected to simulate any more than the general behaviour of salinity over time.

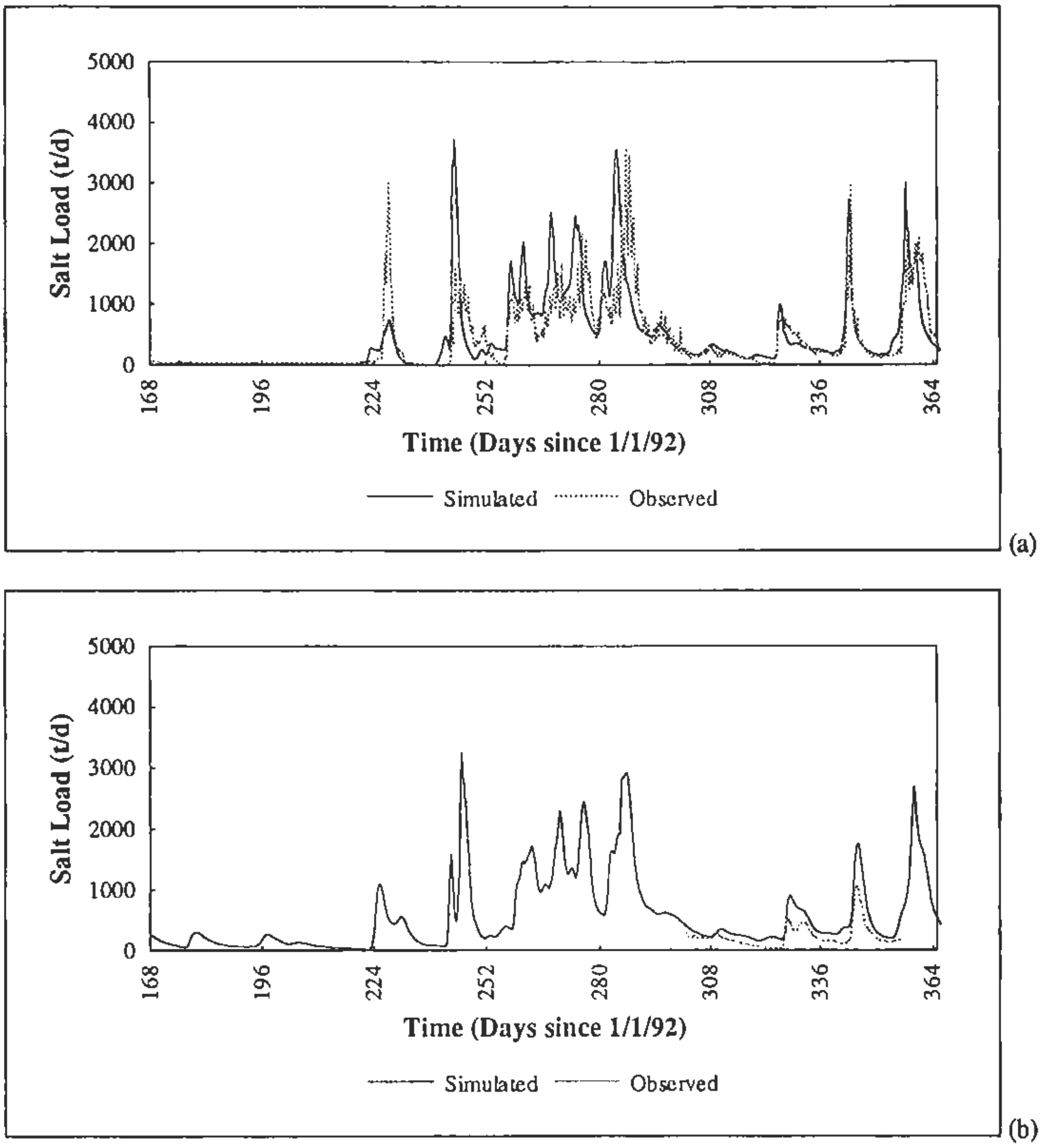
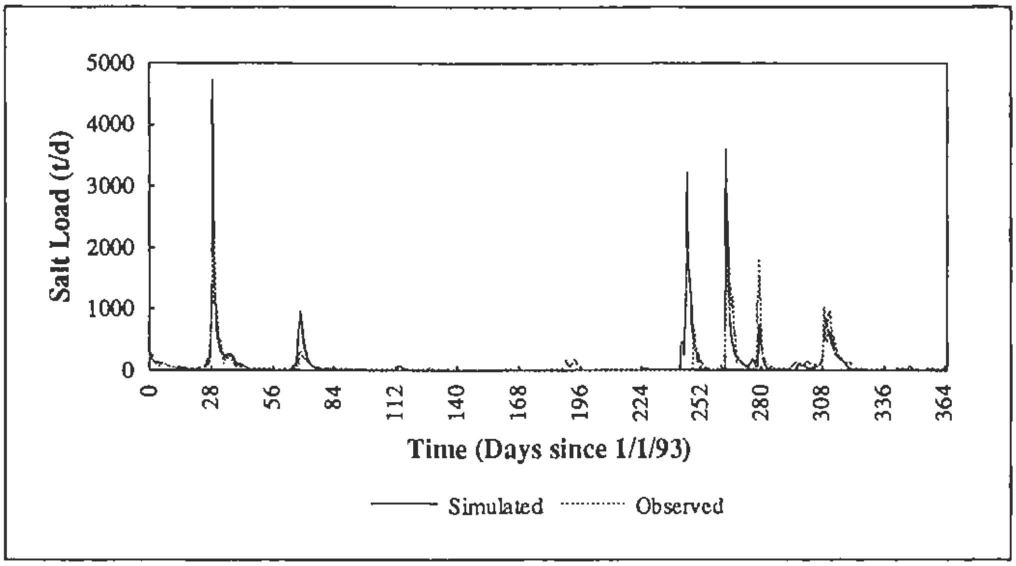


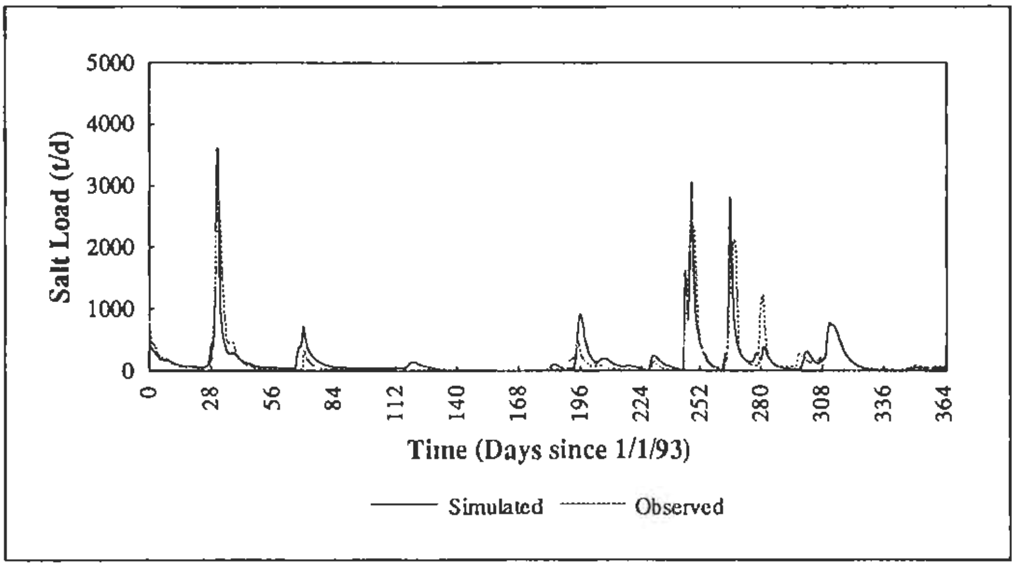
Figure 6.20: Simulated and observed salt fluxes for the Wimmera River at (a) Horsham and (b) Upstream of Dimboola for the period 17/6/92 to 31/12/92. Salinity boundary condition at Glynwylln specified using a solute rating curve.

6.5.5 IMPROVED UPSTREAM CONCENTRATION BOUNDARY.

Improved boundary condition data are available for the upstream model boundary. These improved data consist of a continuously monitored time-series of salinity. Since it is necessary to predict the upstream salinity using a solute rating curve for periods when continuous salinity data are unavailable, it is useful to establish the improvement in the models predictive ability when the better data is used. Figures 6.22 and 6.23 and Tables 6.5 and 6.6 provide a comparison of simulations using the solute rating curve and continuously monitored data for the



(a)



(b)

Figure 6.21: Simulated and observed salt fluxes for the Wimmera River at (a) Horsham and (b) Upstream of Dimboola for 1993. Salinity boundary condition at Glynwylln specified using a solute rating curve.

salinity boundary condition at Glynwylln. While simulated salinities are improved during some periods by using monitored salinities at the Glynwylln Gauge, there is little improvement in the overall predictive ability of the model. It is likely that this is due to the fact that, while one boundary condition has been improved, there is still a significant amount of uncertainty due to poor quality boundary conditions for a number of other surface water and groundwater inflows.

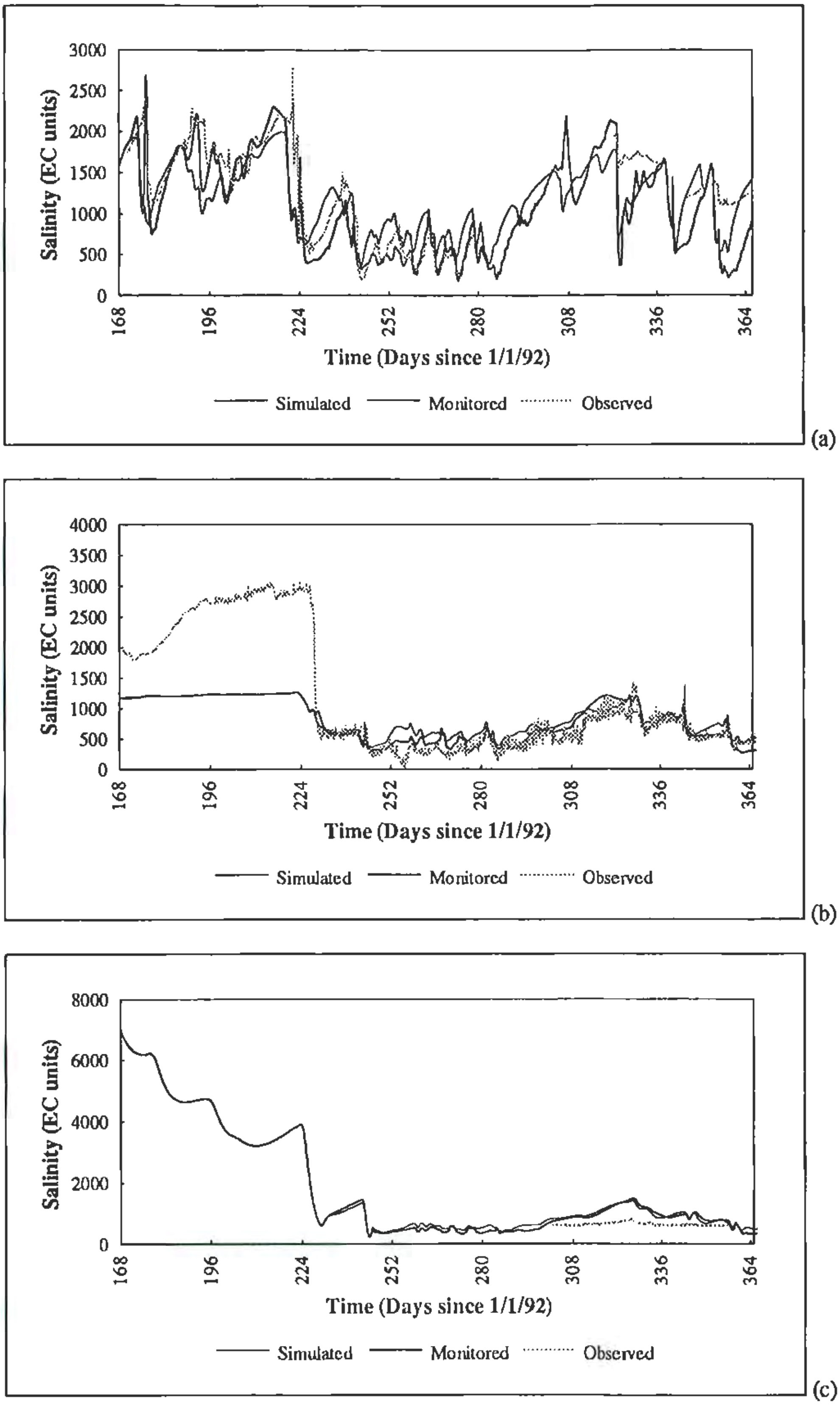


Figure 6.22: Comparison of simulated salinities for the Wimmera River at (a) Upstream of Glenorchy, (b) Horsham and (c) Upstream of Dimboola for the period 17/6/92 to 31/12/92 using solute rating curve and continuously monitored salinity for the boundary condition at Glynwylln. Observed salinity is also included.

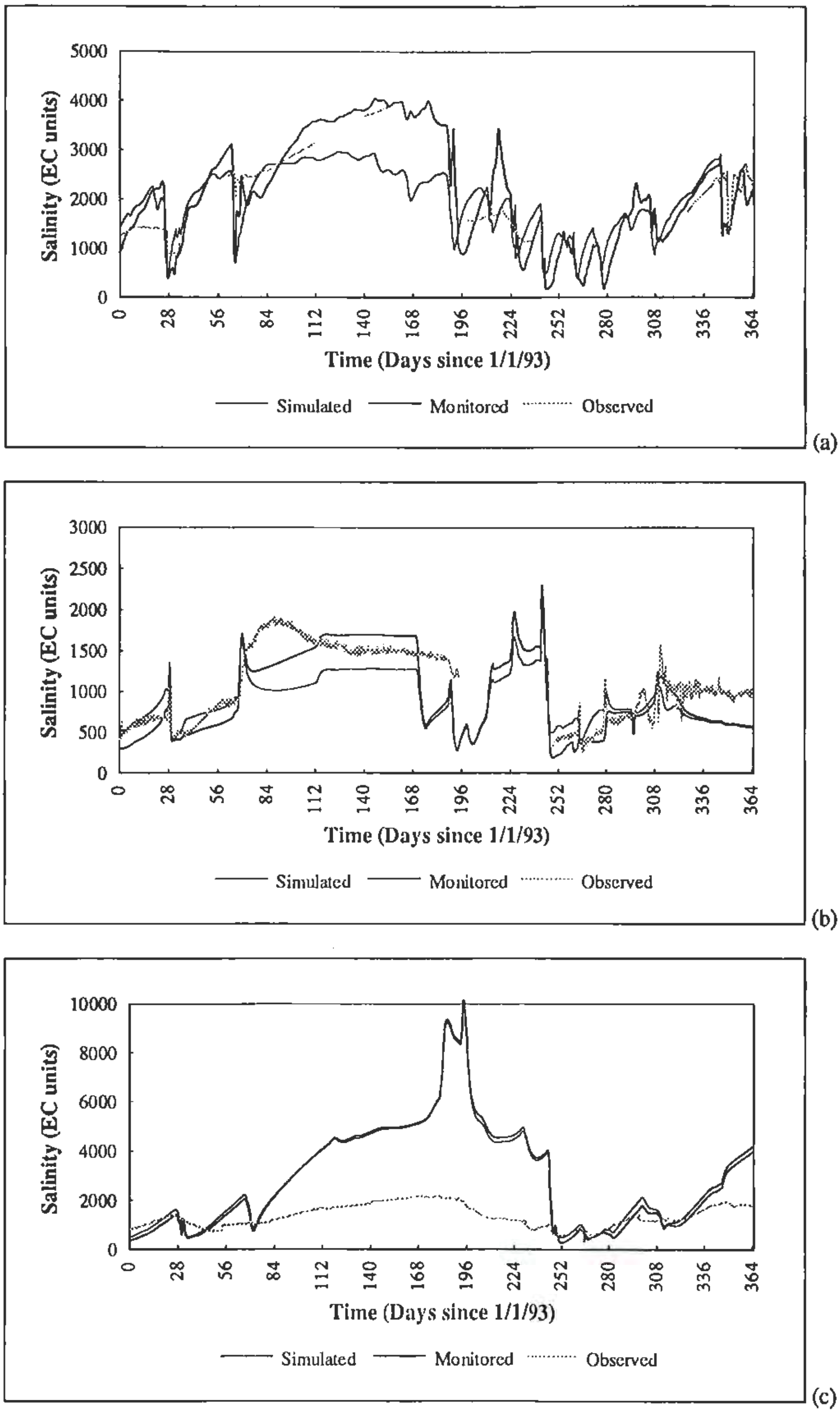


Figure 6.23: Comparison of simulated salinities for the Wimmera River at (a) Upstream of Glenorchy, (b) Horsham and (c) Upstream of Dimboola for 1993 using solute rating curve and continuously monitored salinity for the boundary condition at Glynwylln. Observed salinity is also included.

6.6 SUMMARY AND CONCLUSIONS.

A physically based model of the Wimmera River between Glynwylln and Lochiel has been developed and tested. The MIKE 11 modelling package was used and a number of modifications were made to extend this model.

Flow resistance was calibrated using the rating curves at gauging stations along the river. No trend in flow resistance along the river was evident so a value of 0.06 was selected as appropriate for all discharges and reaches. This simple parameterisation scheme is consistent with the low level of information available. Errors in the celerity of flood waves indicated that the storage in the stream was being underestimated. This is thought to be due to limited knowledge of relict channels and low lying areas adjacent to the stream which increase the storage. A second parameter was introduced which increased the storage in the channel by 50%.

The hydrodynamic model predicts discharge at Glenorchy well while the performance at other stations is variable between events but consistent between stations for a given event. There is a tendency for the model to underestimate both peak and total discharge at downstream gauges and for the wave celerity to be overestimated for large events. A significant proportion of the errors in the simulation of discharge is likely to result from poorly specified boundary conditions for the many ungauged inflows; however the crude manner in which the additional storage effects have been included in the model will also be responsible for some of the error.

The solute transport model is capable of simulating the fluctuations in salinity as discharge varies and the mean salinity level relatively well during moderate to high flows. Errors in predicted salinity can become large at very low or zero discharge. This may be due to errors in prediction of low flows, errors in groundwater inflows or the exclusion of the effects of saline pools. Salt transport in the Wimmera River is dominated by moderate to high flow events. The use of monitored salinity for the boundary condition at Glynwylln has a relatively small impact on the accuracy of simulations and certainly does not lead to significant improvements.

It is recognised that there are a number of limitations associated with the model described in this Chapter. While some limitations have been identified in passing, they have not been discussed in detail. A detailed discussion of the modelling is deferred until Chapter 10 so that a consideration of the modelling of saline pools can be included.

CHAPTER 7 - STRATIFIED FLOW REVIEW.

7.1 INTRODUCTION.

The transportation and transformation of solutes in the environment. Density stratification is an important feature of environmental systems because it has a major impact on the movement of solutes through those systems. Turbulence and vertical mixing are suppressed by stable density stratification (Taylor, 1927, 1931; Sherman et al., 1978; Hossain and Rodi, 1980; Hannoun and List, 1988), therefore the impact of stratification on mixing must be considered when studying the behaviour of water quality in such systems (Hannoun and List, 1988; Narimousa and Fernando, 1987; Fernando, 1991). While the rate of vertical transport of solutes is of particular interest when attempting to model water quality in a stratified system, a modeller should also understand the processes involved in this transport. This review will therefore consider both the processes involved in vertical mixing in stratified systems and the rate of vertical mixing that may be expected in a given set of circumstances.

The literature relating to the behaviour of density stratified fluid systems is very large and rapidly expanding (Thorpe, 1987). Therefore this review is limited to those portions of the literature of relevance to Saline Pools.

7.2 DENSITY STRATIFICATION.

Stable density stratification may result from variations in temperature, solute concentration or sediment concentration over the water column. Examples of such stratification include the thermal stratification of the ocean (Price et al., 1978; Kato and Phillips, 1969; Fernando, 1991), atmosphere (Taylor 1927, 1931; Macagno and Rouse, 1961) and lakes (Bloss and Harleman, 1980; Spigel and Imberger, 1980; Harleman, 1982; Christodoulou,1986); salinity stratification of estuaries (Fischer et al., 1979; Christodoulou, 1986) and thermal stratification due to cooling water plumes from power stations (Hossain and Rodi, 1980; Fernando, 1991). Some Australian rivers, including the Wakool (Herat, 1984), the Murray (Western Australia) (Morrissy, 1979), the Barwon (Roderick, 1988), the Campaspe (McGuckin,1990), the Richardson, Loddon, Little Murray, Avoca and Glenelg (McGuckin et al., 1991), and the Wimmera (Anderson and Morison, 1989c; 1989g) contain density stratified pools during parts of the year. This density stratification arises primarily from vertical variations in salinity.

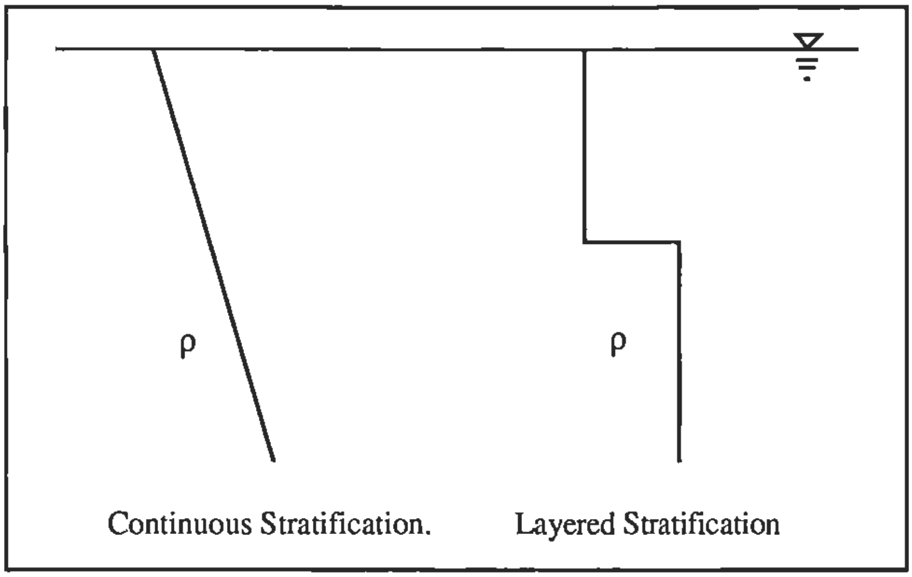


Figure 7.1: Types of density stratified systems.

There are two types of density profile found in stratified systems. The first is a profile where density increases continuously with depth (Figure 7.1). This is often referred to as continuous stratification. The second is characterised by layers of approximately constant density separated by thin interfaces across which there is a sharp density jump (Figure 7.1). The terms layered system and stratified system will be used interchangeably to refer to the second type of stratification. There is also a fundamental difference between stratified systems which are subject to a shear-flow and those which are not (Fernando, 1991). The stratified pools in the Wimmera River tend to have a sharp density interface (Anderson and Morison, 1989c) and, when the river is flowing, a layer of fresh water flows over a saline layer which is contained in the bottom of a scour depression. The lower layer therefore must be either quiescent or circulating within the scour depression.

7.2.1 PARAMETERISATION OF STRATIFIED SYSTEMS.

An understanding of the parameterisation of stratified flows is useful for the following discussion. The major parameter required to describe a layered system subject to a shear-flow is the Richardson number. In general the Richardson number used in layered systems can be expressed as $Ri = \frac{g'l}{u^2}$ where $g' = g\frac{\Delta\rho}{\rho}$ is a reduced value of gravity, more often referred to as the buoyancy, and u and l are velocity and length-scales which depend on the particular flow situation. A shear-free interface subject to turbulence could be described using the integral length and velocity-scales of the turbulence (Hannoun and List, 1988). In shear-flows several possible velocity-scales exist including the shear velocity, mean layer velocity and velocity difference between the two layers. The layer depth, interface thickness and shear layer thickness are possible length-scales. If one layer is quiescent then the mean layer velocity and interfacial velocity jump are equivalent.

An important vertical mixing process in many stratified shear flows is entrainment. Entrainment is fundamentally a turbulent process (Price et al., 1978) so it should be possible to examine the sources of turbulence to find appropriate scales to describe the flow. Velocity shear would be expected to be the major source of turbulence (Turner, 1973). Furthermore turbulence produced within the interface would be expected to entrain fluid more efficiently than that produced externally at the walls (Turner,1973). Experimental observations of turbulence produced near a wall show that the turbulence largely dissipates near the wall (Hinze, 1975) and therefore is unlikely to have the opportunity to entrain fluid across the interface. A number of authors (Pollard et al., 1973; Price, 1979; Narimousa and Fernando, 1987; others) have argued that entrainment is due to shear instability at the interface and that therefore the relevant velocity-scale is the interfacial velocity jump. If one layer is quiescent it becomes reasonable to use the mean layer velocity as the velocity-scale.

The appropriate length-scale for shear-flow could be the thickness of the shear layer, δ_u, the thickness of the interface, δ, or the thickness of the turbulent layer, h. The definitions of these thicknesses vary from author to author and they are used here more as conceptual properties rather than physically defined properties. They can be considered as the height over which the velocity is changing rapidly, as the height over which the density is changing rapidly and as the depth of the mixed layer respectively. Lofquist (1960) and Narimousa and Fernando (1987) have examined the thickness of the interface and the thickness of the shear layer and found that both were proportional to the turbulent layer depth. Given that the mixed layer depth is the only one of these three lengths defined in terms of bulk flow properties it will be used initially.

A two layer flow system similar to that found in the Wimmera River is shown in Figure 7.2. The upper layer is moving at velocity U, has a depth h and density ρ. The lower layer is quiescent and has density $\rho+\Delta\rho$. The rate of entrainment is U_e and gravity is g. The buoyancy is $g'=g\frac{\Delta\rho}{\rho}$. The relevant parameters are the entrainment parameter $E=\frac{U_e}{U}$ and the Bulk Richardson number $Ri_b=\frac{g'h}{U^2}$. The Bulk Richardson number represents the ratio of the buoyancy and inertial forces (Turner, 1973). For those more comfortable with Froude numbers, the Bulk Richardson number is simply the square of the inverse of the densiometric Froude number.

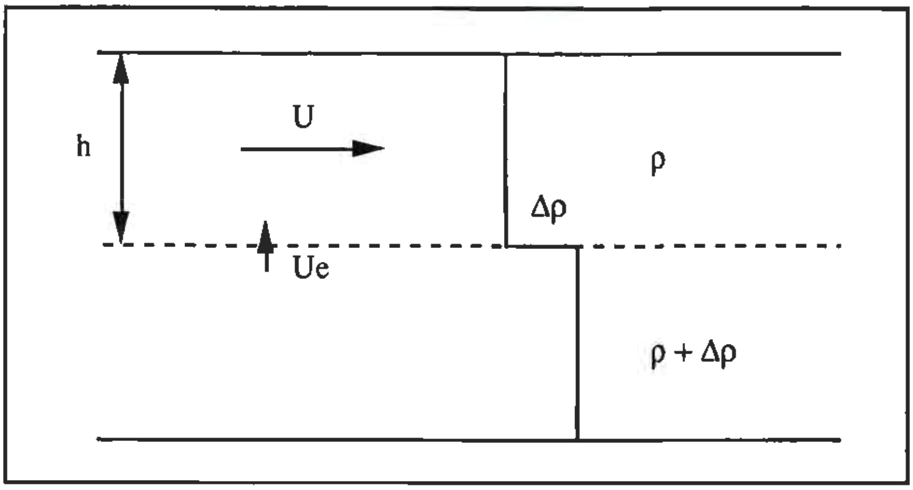


Figure 7.2: Variables describing a two layer density stratified system subject to a buoyant overflow.

Another parameter often used to describe stratified shear flows is the Gradient Richardson number, $Ri_g = -g\frac{\partial\rho}{\partial z}/\rho\left(\frac{\partial u}{\partial z}\right)^2$ (Turner, 1973). This is an internal parameter related to the relative strength of buoyancy and shear effects and as such is more closely related to the turbulence production in stratified shear flows. However, as Ri_g is not defined in terms of bulk flow properties, Ri_b is preferred for this study.

7.3 MIXING PROCESSES.

A brief qualitative description of several mixing mechanisms that can operate in stratified systems is given in this section.

7.3.1 TURBULENT ENTRAINMENT.

Early observations of mixing over a stable density interface subject to a velocity shear were made by Taylor (1927). With a lighter layer flowing over a quiescent pool of brine, Taylor observed the continual removal of a small amount of brine at low velocity. As the velocity exceeded a certain value rapid mixing suddenly occurred. Taylor also conducted the same experiment using a dyed pool of water. In this case the interface at the upstream limit of the pool was similar to the stratified case, however mixing gradually increased downstream.

In this simple experiment two important effects of a stable density discontinuity are demonstrated. Firstly the stable density profile acts to suppress vertical mixing and secondly it allows a new mechanism whereby the flow can become unstable and produce turbulence internally. Vertical mixing associated with this turbulence, and in some cases turbulence produced elsewhere, is referred to as turbulent

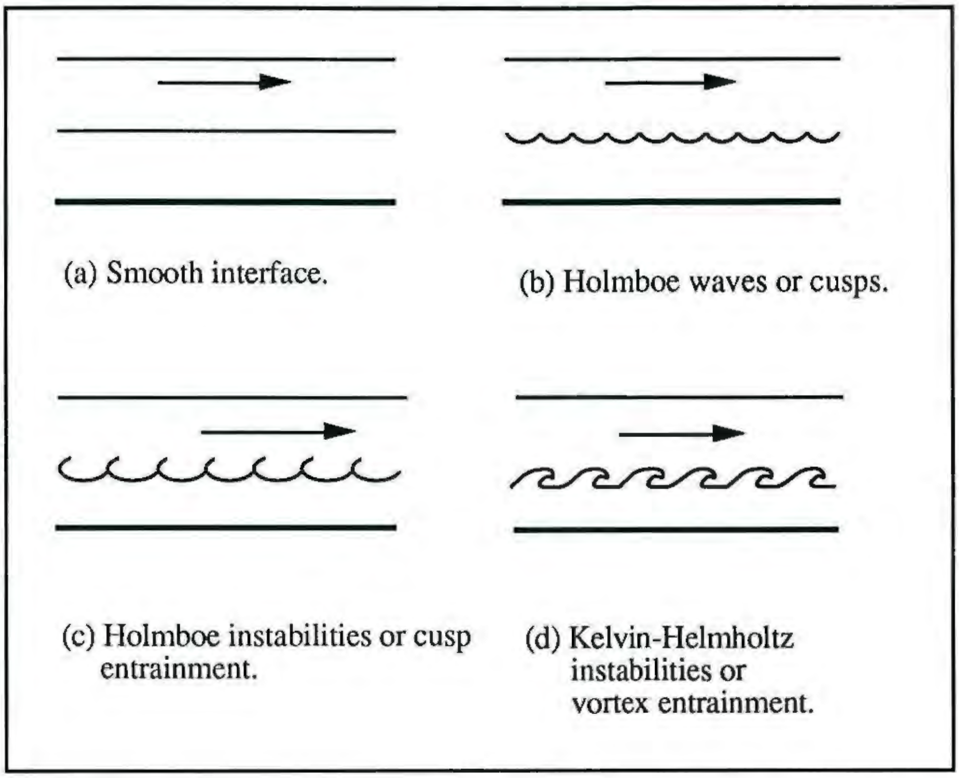


Figure 7.3: Interfacial characteristics of stratified shear flows. Richardson number decreases from (a) to (d).

entrainment. Many experiments aimed at studying the mixing processes and determining the rate of mixing across a density interface have since been published.

When a light layer flows over a denser layer, a sharp interface is present at high Richardson numbers (Figure 7.3a) (Keulegan, 1949). As the velocity of the upper layer increases the Richardson number decreases and symmetric waves appear at the interface and travel downstream (Figure 7.3b). These waves have become known as Holmboe waves (Christodoulou, 1986) due to a study of symmetric waves by Holmboe (1962).

When the Richardson number is decreased further, these waves become sharp crested and begin ejecting fluid from their crests into the turbulent layer thus leading to mixing (Figure 7.3c) (Keulegan, 1949; Rouse and Dodu, 1955; Kato and Phillips, 1969; Moore and Long, 1971). This mixing process is one of the two fundamental mixing processes postulated by Pedersen (1980) and is often referred to as cusp entrainment because the interface has a cusp-like appearance. It is thought that the mixing results from mechanical stirring by eddies in the turbulent layer (Moore and Long, 1971; Spigel and Imberger, 1980). Stephenson and Fernando (1991) observed intermediate density layers within the interface which are thought to result from collapse of isolated mixed regions (Wu, 1973;

Stephenson and Fernando, 1991). Thorpe (1987) notes that surprisingly little is known about Holmboe waves and their instability except that they have a billow-like structure and their crests are generally short and aligned perpendicular to the flow. The effective mass flux across the interface is small.

As the flow velocity is increases further, Kelvin-Helmholtz instabilities form. These have been extensively studied and are reviewed by Thorpe (1987) and Fernando (1991). Kelvin Helmholtz instabilities consist of parallel concentrated vortices which are oriented perpendicular to the flow and are located on the turbulent side of the interface (Figure 7.3d) (Fernando, 1991). This entrainment is referred to as vortex entrainment by Pedersen (1980) who regards it as the second fundamental entrainment phenomenon.

From the above description it is apparent that there may be several different mixing mechanisms involved in turbulent entrainment at density interfaces. It might therefore be expected that several different entrainment regimes would exist.

7.3.1.1 Direction of Transport.

Two types of mixing are possible in stratified systems subject to shear flow. Entrainment mixing occurs where only one layer is turbulent and turbulent diffusion occurs when both layers are equally turbulent. Between the two extremes mixing is a combination of entrainment and turbulent diffusion (Carstens, 1970, Pedersen, 1980).

Entrainment is a one way process in which mass is transported into the turbulent layer and the interface migrates away from the turbulent layer. The one-way nature of entrainment is due to the fact that fluid ejected from the interface must be removed from the interface and mixed across the layer. This can only occur in the presence of turbulence. Therefore if only one layer is turbulent the transport must be a one way process (Carstens, 1970; Pedersen, 1980).

When both layers are equally turbulent, fluid ejected from the interface on either side is removed at the same rate and a two way transport process without interface migration occurs. This has been observed by Crapper and Linden (1974), Moore and Long (1971) and Macagno and Rouse (1961) among others. When the level of turbulence in the two layers is different, the transport rate from the less turbulent layer to the more turbulent layer will exceed that in the opposite direction. Thus there will be a net transfer to the more turbulent layer and the interface will migrate away from the more turbulent layer. This is generally the case in estuaries with layered stratification (Carstens, 1970; Grubert, 1980, 1989). This situation can be represented by a combination of entrainment and turbulent diffusion and it has been suggested that the rate of transfer to each layer can be calculated independently by assuming that the entrainment laws apply (Christodoulou, 1986).

7.3.2 MIXING FROM A SQUARE CAVITY.

A recent study that aimed to examine specifically the flushing of saline pools is considered here. Armfield and Debler (1993) conducted a series of laboratory investigations of the flushing of a square cavity, initially filled with saline water, by a fresh over flow. These experiments were conducted to validate a numerical model based on the Navier-Stokes equations.

The experiments were conducted by filling the cavity with saline water; then rapidly starting a flow in the upper layer. Three features of the flow developed in rapid succession and removed a significant amount of saline water from the cavity. The overflow initially pushed a large splash of saline water out of the downstream end of the cavity. A vortex then formed which mixed some of the upper layer water into the lower layer forming a layer of intermediate density and leading to a secondary splash which removed more saline water from the cavity. Continued seiching then ejected water from the intermediate density layer out of the cavity. After these initial stages, a circulatory motion in the upper part of the cavity slowly transported the remaining dense fluid from the cavity. These experiments were conducted with Richardson Numbers (based on the entrance velocity and cavity height) of 0.58 and 0.34 and numerical simulations were conducted for these two plus four other cases. The numerical simulations were able to reproduce successfully the major features of the flow.

There are likely to be significant differences between Armfield and Debler's (1993) experiments and flushing of saline pools in the Wimmera River. Firstly, the three initial stages of development of the flow, which are responsible for significant mixing, appear to be the product of internal seiching resulting from the rapid build up of flow. The time-scale for dissipation, T_d, of an internal seiche with period $T_i = 2L_B/\sqrt{g' h_1 h_2/(h_1+h_2)}$ can be estimated using an energy balance (Spigel and Imberger, 1980). L_B is the length of the basin and h_1 and h_2 are the depths of the upper and lower layers respectively. The work done within an oscillatory boundary layer is equal to its kinetic energy content per time period. Hence in time, t, the work done is $\frac{1}{2}\delta_b A_b \rho u^2 t/T_i$ where δ_b is the boundary layer thickness, A_b is the boundary area and u is the velocity-scale of the wave. The kinetic energy which must be dissipated by the boundary layer is $\frac{1}{2}V_B \rho u^2$, where V_B is the volume of the basin. Equating these two energies then gives a dissipation time-scale of:

$$T_d = \frac{T_i V_B}{\delta_b A_b} \quad (7.1)$$

The thickness of a turbulent oscillatory boundary layer is given by (Kalkanis, 1964):

$$\delta_b = U_{max} T_i^{0.5} k / 471 \nu^{0.5} \quad (7.2)$$

where k is the roughness height, U_{max} is the maximum velocity due to the oscillations and ν is the kinematic viscosity.

For typical saline pools found in the Wimmera River L_B = 100 m, g' = 0.1 m^2/s, h_1 = 4 m and h_2 = 1 m giving T_i = 700 s. Chow (1959) gives roughness heights of between 0.03 m and 0.9 m for natural rivers. Taking k = 0.1 m as an appropriate value for a river with relatively fine bed material and estimating U_{max} as the velocity of the upper layer U_1 = 0.05 m/s implies δ = 0.28 m. The dissipation time-scale is then of the order of 3 hours. However hydrographs at Horsham rise over a time-scale of 1-2 days which implies that the discharge in the river changes slowly compared to the dissipation time-scale for an internal seiche. It is therefore unlikely that the rapid removal of fluid observed at the start of the experiments would be significant in the river.

Secondly, these experiments were conducted at Richardson Numbers that are two orders of magnitude lower than those observed in the Wimmera River. Armfield and Debler (1993) showed that the initial splash that led to a significant amount of mixing increased significantly as the Richardson Number decreased. Therefore the difference in Richardson Numbers is important since it indicates that any initial mixing in the Wimmera River is likely to be significantly less than that observed in the experiments.

Thirdly, a square cavity was used in the experiments. However, in the river the entrance and exit to scour depressions slope at small angles (less than 10°) to the horizontal. In the experiments the flow separates from the boundary at the entrance to the cavity and drives a circulation within the cavity. This results in water from the dense layer being transported towards the upstream end of the cavity and subsequently up into the intermediate density layer and out of the cavity. Because the expansion of the flow in the river is gradual such a separation is much less likely to occur in which case the circulation would not develop.

While Armfield and Debler's experiments are unlikely to be applicable to saline pools, their results do indicate that two-dimensional (vertical and longitudinal) effects may be important in the flushing of saline pools. A series of experiments conducted by Nolan (1994) as part of this project are described in Chapter 8. These experiments were designed to overcome the above criticisms of Armfield and Debler's (1993) experiments.

7.3.3 CONVECTIVE MIXING.

7.3.3.1 Penetrative Convection.

When the surface of a water body cools, the water becomes denser and tends to sink. If the resulting convective motions are sufficiently vigorous they can penetrate into an underlying stable part of the water column. This is known as penetrative convection and is associated with vertical mixing (Turner, 1973). Lakes which are sheltered from surface winds typically have an epilimnion (surface layer) which becomes thermally stratified due to daytime heating and then convectively mixes as the surface cools at night. Convective mixing also occurs during seasons when strong surface cooling occurs (eg. autumn in Victoria) (Fischer et al., 1979). Given the nature (still, deep, relatively well sheltered) of saline pools in the Wimmera River and the possibility of natural transfers of thermal energy between the river and the atmosphere, similar processes could contribute to mixing in the Wimmera River.

7.3.3.2 Double-Diffusive Effects.

Convective processes can be modified by the presence of more than one property, for example salinity and temperature, which influence the density structure. In the Wimmera River density changes occur with depth due to changes in both salinity and temperature (Anderson and Morison, 1989c). Anderson and Morison's data show that the density profile due to salinity changes is stable and results in density differences significantly greater than those due to the temperature differences in most cases. Both stable and unstable temperature profiles can be found, in association with stable salinity profiles, in the stratified pools.

Where two properties are simultaneously causing changes in density, vertical mixing is influenced by the relative diffusivities of the two properties. If the distribution of one of the properties is unstable then enhanced vertical mixing due to the release of the potential energy associated with the unstable component is possible. When the stratification is stable with respect to the property with the lower diffusivity, for example salt rather than heat, it is known as a diffusive system. In diffusive systems turbulent convecting layers separated by sharp interfaces can form. Such convection can occur in systems where warm salty water is found below cold fresh water when the net density profile is unstable or marginally stable (Turner, 1973).

Unstable temperature profiles associated with stable salinity profiles are found in the Wimmera River on some occasions. However the density differences due to the salinity variations are often significantly greater than those due to temperature variations in which case diffusive convection would not be expected.

7.4 ENTRAINMENT RELATIONSHIPS.

Although turbulent entrainment is not the only process which may be important in the flushing of saline pools, an understanding of entrainment processes and likely entrainment rates is important when interpreting mixing rates observed during flow events. Entrainment relationships that have been proposed are now discussed.

Many experimental studies of stratified systems have been undertaken with the aim of understanding the stratified flow phenomena and quantifying the rate of mixing in stratified systems. Results from experiments have provided inspiration for theoretical developments and these results are described where relevant. The qualitative observations made during entrainment experiments (§7.3) suggest that a number of different mixing regimes exist which depend on the Richardson number and Peclet number of the system. These are considered in turn below.

7.4.1 MOLECULAR DIFFUSION.

A lower limit on the buoyancy flux across a density interface due to the effect of molecular diffusion must exist (Atkinson and Munoz, 1988; Hannoun and List, 1988; Turner, 1973). Several experimenters have examined the importance of molecular diffusion under different conditions. The Peclet Number, $Pe = \frac{Uh}{K}$, becomes important when molecular diffusion makes a significant contribution to vertical mixing (Turner, 1973). Pe tends to be large if salt is the stratifying medium and low when heat is the stratifying medium. Turner (1968) found that, compared with experiments using salt as the stratifying medium, mixing was significantly more rapid when heat was the stratifying medium. Hopfinger and Toly (1976) conducted a series of high Peclet number experiments in which Peclet numbers were 20 times those of (also high peclet number) Turner (1968). Comparing the two sets of experiments, Hopfinger and Toly found no dependence on Peclet number which also suggests that molecular diffusion is not important at high Peclet numbers. Although molecular diffusion implies a lower limit on the buoyancy flux at high Ri and Pe, that limit is small and does not significantly modify other mixing processes. Narimousa et al. (1986) suggest that the buoyancy flux across salt stratified sheared density interfaces in dominated by molecular diffusion when $Ri_b > 20$.

Several experimental and theoretical studies have examined the role of molecular diffusion in interfacial mixing at high Ri and low Pe numbers (eg. Fernando, 1986; Atkinson and Munoz, 1988; Hannoun and List, 1988; Mory, 1991). These studies are not pursued here since they are of limited relevance to this study.

7.4.2 HIGH RICHARDSON NUMBER REGIME.

At high Richardson and Peclet numbers (salt-stratification) in both grid-stirred and shear-flow experiments the entrainment can be represented by $E \propto Ri^{-n}$, where n=3/2 (Turner, 1973; Hopfinger and Toly, 1976; Kit et al, 1980; Deardorff and Yoon, 1984; Hannoun and List,1988). If a simple energy analysis is applied a value of n = 1 can be derived which is inconsistent with experimental observations in the high Richardson number range. This inconsistency motivated Linden (1973) to study the interaction between an interface and an impinging vortex in a salt-stratified system.

Linden observed an impact stage during which the interface deflected and the vortex ring flattened, then a recoil during which the vortex collapsed and denser fluid was ejected into the upper layer and mixed. As the Richardson number increased the interface deflection decreased and the vortex flattening increased. By assuming that the kinetic energy associated with the vertical motion of the vortex is temporarily stored in the interface and then converted to potential energy of the system during the recoil stage, Linden calculated an entrainment volume for each vortex. Then assuming that the turbulence in the mixed layer could be represented by an ensemble of vortex rings, Linden showed that $E \propto Ri^{-3/2}$. As the Richardson number decreases Linden's assumed interface profile at the point of maximum distortion is incorrect and the model breaks down (Linden, 1973).

Long (1978) used Hunt and Graham's (1978) rapid distortion theory to derive an alternative model of entrainment at high Richardson numbers. Hunt and Graham (1978) studied turbulence near a flat plate and discovered that the turbulence normal to the plate was suppressed and the turbulence parallel to the plate was amplified. Using Hunt and Graham's results, Long (1978) proposed that $E \propto Ri^{-7/4}$ which implies a slightly slower entrainment rate than the experimental evidence suggests. Hannoun et al. (1988) observed the turbulence structure near the interface during a series of grid stirred experiments. While the results were in qualitative agreement with the rapid distortion theory of Hunt and Graham (1978), the amplification and suppression rates differed. Using the results of Hannoun et al., Hannoun and List (1988) modified Long's (1978) analysis to obtain E α $Ri^{-3/2}$.

7.4.3 INTERMEDIATE RICHARDSON NUMBER REGIME.

Many authors (Kato and Phillips, 1969; Wu, 1973; Moore and Long, 1971; Buch, 1981; Narimousa and Fernando, 1987; others) have reported experimental entrainment relationships of the form E α Ri^{-1}. It is possible to explain these results by considering the Turbulent Kinetic Energy (TKE) budget. When entrainment occurs TKE is used to increase the TKE of the entrained fluid and to raise the potential energy of the fluid system. TKE is also dissipated through the action of molecular viscosity and generated in a number of ways. Generation of TKE may occur through the action of a surface stress, as a product of velocity shear or as a result of an imposed buoyancy flux, particularly as a result of surface cooling or bottom heating (Pollard et al., 1973; Ottesen-Hansen, 1975; Sherman et al., 1978; Bloss and Harleman, 1980; Spigel and Imberger, 1980). If the TKE budget is dominated by a balance between the production of TKE and the increase in potential energy of the system then an entrainment law of the form E α Ri^{-1} can be shown (Rouse and Dodu, 1955).

7.4.4 LOW RICHARDSON NUMBER REGIME.

Entrainment is not explained by E α Ri^{-1} at low Richardson numbers (Pedersen, 1980). At very low Richardson numbers or high Densiometric Froude numbers rapid mixing has been observed by several writers (Ellison and Turner, 1959; Chu and Vanvari, 1976; Anwar and Weller, 1981). These observations have all been made when a buoyant overflow enters a two layer region. At Richardson numbers close to zero Chu and Vanvari observed that the flow initially behaved as a neutral wall jet entraining fluid at a constant rate given by E = 0.036. Both Chu and Vanvari (1976) and Anwar and Weller (1982) reported rapid decreases in mixing downstream as the mixed layer depth increased. This suggests that there may be some critical flow condition or Richardson number below which mixing rapidly increases.

Stability criteria for layered flow systems have been often studied (see Holmboe, 1962; Orlanski and Bryan, 1969; Howard and Maslowe, 1973; Frankignoul, 1972; Abarbanel et al., 1984; Lawrence et al., 1991). The classic result of this work is the Miles-Howard theorem which predicts that unbounded parallel shear-flows are always stable to infinitesimal disturbances if the Gradient Richardson number is everywhere greater than 1/4 (Howard and Masloe, 1973; Miles, 1986; Fernando, 1991). Many other analyses have been carried out which provide support for the assumption of a critical Gradient Richardson number.

At low Richardson numbers the kinetic energy required to accelerate the entrained fluid is significant compared to the increase in potential energy of the system. Several authors (eg. Pollard, 1973; Kundu, 1981; Kranenburg, 1981) have successfully developed models of the deepening of the ocean mixed layer during storms. These models assume a dynamic equilibrium for the mixed layer and are capable of simulating rapid mixing events. Momentum is supplied to the mixed layer by wind shear and water with zero momentum is entrained from below the thermocline when the Richardson number falls to a critical value.

7.5 GENERAL ENTRAINMENT LAWS.

It is evident from the above discussion that entrainment in density stratified shear-flows cannot be explained by a single process. There are several different regimes of mixing across sheared density interfaces and many experimental studies have been conducted over different ranges of Richardson number and with different flow configurations. Several different forms of the Richardson number, which is the fundamental parameter describing such flows, have been used in the literature. It would seem difficult under such circumstances to develop a unified view of entrainment in density stratified shear-flows; however two approaches have been tried with some success.

Christodoulou (1986) used available empirical results and the theoretical functional forms for entrainment laws discussed above, to develop a unified entrainment relationship based on the Bulk Richardson number. Using the Bulk Richardson number has the advantage of simplicity since mixing is predicted from bulk layer properties. The entrainment law proposed by Christodoulou (1986) has four parts each of which apply to a specific Richardson number range as follows:

$$E = 0.07, \qquad Ri_b < 0.01 \qquad (7.3a)$$

$$E = 0.007\, Ri_b^{-1/2}, \qquad 0.01 < Ri < 0.1 \qquad (7.3b)$$

$$E = 0.002\, Ri_b^{-1}, \qquad 0.1 < Ri < 10 \qquad (7.3c)$$

$$E = 0.007\, Ri_b^{-3/2}, \qquad Ri > 10 \qquad (7.3d)$$

The above relationships have been fitted to the results of surface stress experiments by Kato and Phillips (1969), Wu (1973), Kantha et al. (1977), Kit et al. (1980); buoyant overflow experiments of Ellison and Turner (1959), Chu and Vanvari (1976), Pedersen (1980) and Buch (1981); experiments with density currents by

Ellison and Turner (1959) and Lofquist (1960); and counter-flow experiments by Macagno and Rouse (1961) and Moore and Long (1971). Experimental results were presented in terms of the Bulk Richardson number defined using the turbulent layer depth and the velocity jump across the interface. The entrainment parameter was expressed as the entrainment velocity divided by the mean layer velocity. Where necessary (eg. the surface stress experiments) the experimental results were expressed using Ri_b by assuming relationships between the original Richardson number and Ri_b based on reported observations.

Another approach that can be used to develop a general entrainment law is to consider the TKE available to the flow. This has been done by Ottesen-Hansen (1975), Sherman et al. (1978), Spigel and Imberger (1980), Pedersen (1980) and others. Atkinson and Munoz (1988) have considered the situation where TKE is produced by velocity shear and used to increase the potential energy of the system. Fluxes due to entrainment and molecular diffusion are included. Equation 7.4 represents the TKE budget for a unit mass of fluid.

$$\frac{\partial q}{\partial t} = C_f \frac{u^3}{h} - \frac{g}{\rho} \overline{w'\rho'} \tag{7.4}$$

The term on the left represents the time rate of change of TKE per unit mass and can be parameterised as $C_t U^2 U_e/h$. The first term on the right is the shear production and the second term is the work against gravity which depends on the buoyancy flux. Atkinson and Munoz (1988) have included both the buoyancy flux due to entrainment and that due to molecular diffusion. The buoyancy flux due to entrainment is simply $-g'U_e$ and the buoyancy flux due to molecular diffusion is $-C_d g'(KU/h)^{1/2}$ (Atkinson and Munoz, 1988). Substituting these parameterisations into 7.4 results in,

$$C_t U^2 U_e/h = C_f \frac{u^3}{h} + g'U_e + C_d g'(KU/h)^{1/2}, \tag{7.5}$$

which can be rearranged to give

$$E = \frac{C_f - C_d g'h/U^2 (K/Uh)^{1/2}}{C_t + g'h/U^2}, \tag{7.6}$$

or

$$E = \frac{C_f - C_d \, Ri \, Pe^{-1/2}}{C_t + Ri} \qquad (7.7)$$

Atkinson and Munoz (1988) have successfully used this relation to model entrainment experiments where the system was driven by a combination of shear-flow and bottom heating. Atkinson (1988) also demonstrates that the energy model can adequately describe the data used by Christodoulou (1986). The close agreement between Atkinson (1988) and Christodoulou (1986) is not surprising since the two equations are fitted to the same data. Values of the coefficients are not provided for general applications.

There are two models which are able to explain observed laboratory data relatively well. Christodoulou's (1986) model provides entrainment estimates which are based on several independent laboratory studies and do not require setting of any parameters. This model provides a useful benchmark for comparing observed mixing rates with typical turbulent entrainment rates available from the literature.

7.5.1 SCALE, GEOMETRY AND ROUGHNESS.

Several studies have been conducted which examine the effect of various parameters on the mixing relationship. These are now discussed briefly.

Grubert (1989) has suggested that scale effects may have a significant impact on entrainment rates; however few data support this contention unambiguously. Data collated by Christodoulou (1986) cover a range of experimental scales from natural fjords (Buch, 1981) to small laboratory channels (10 x 25 cm) (Chu and Vanvari, 1976) and do not exhibit any obvious scale effects. Furthermore, Grubert provides average data for each of his experiments which allows the Bulk Richardson number to be calculated. Figure 7.4 shows Grubert's experimental results. Grubert (1989) conducted experiments in 300 mm and 600 mm flumes which were artificially roughened. Although the Richardson number ranges are different, the two ranges are continuous and there does not appear to be to be a difference between the 300 mm and the 600 mm flumes. However it should be noted that the difference in scale is small and that roughening the flumes may disguise any scale effect since roughness also affects the mixing.

The issue of channel roughness has been considered by Jones and Mulhearn (1983) and Grubert (1989). Both these studies demonstrated that entrainment rates increase with increasing channel roughness. While channel roughness does affect the entrainment rate it is apparently of secondary importance in that it shifts the E vs Ri curve up or down rather than changing the basic form of the relationship.

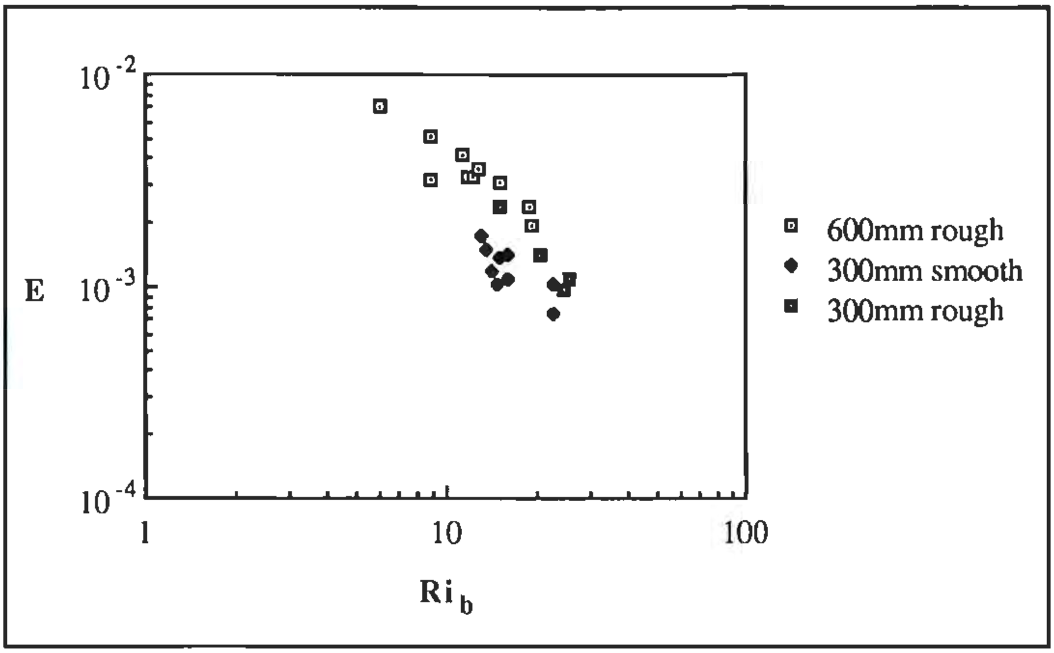


Figure 7.4: Grubert's (1989) experimental results showing the effect of roughness and scale.

Sumer and Fischer (1977) conducted a series of experiments aimed primarily at quantifying the rate of transverse mixing in stratified flows. They used a regular channel and a channel with large-scale irregularities (waves) in one wall. In the irregular channel enhanced vertical mixing was observed in the vicinity of the wavy wall. This prompted Sumer and Fischer to speculate that there may not be a general relationship for vertical mixing in natural streams. Grubert (1989) speculates that a relationship could be found if friction factors were considered. However the two-dimensional nature of the flow around the irregularities would appear to be the primary cause of enhanced mixing and it would seem unlikely that this would be well described by friction factors. Unfortunately no detailed work has been carried out on this problem.

7.6 MODELLING OF STRATIFIED SYSTEMS.

If the vertical movement of solutes in a fluid system is to be modelled the vertical dimension must be included in the model. If a numerical model is to be used, then the system must be discretised vertically. The simplest possible vertical discretisation consists of two layers of variable thickness. At the other extreme, a highly detailed vertical discretisation with vertical transport modelled as a gradient transport process could be used. In such models vertical diffusivities are predicted using local flow characteristics and turbulence models - see, for example, Mellor and Durbin (1975) and Hossain and Rodi (1980).

Two-layer models are relatively simple and easy to verify compared with multilayer models. They also provide a much more realistic picture of the movement of solutes than depth averaged models when strong stratification is present (Christodoulou and Connor, 1980). Two-layer models have been used successfully in canals (Roelfzema, 1980), estuaries (Vreugdenhil, 1970; Hodgins et al., 1977), fjords (Hodgins, 1978) and the ocean (Pollard et al. 1973; Price et al., 1978; others) where they capture the overall characteristics of the system especially when the stratification tends to be characterised by sharp interfaces. A two-layer model applicable to saline pools has been successfully developed as part of this study and is described in Chapter 9.

7.7 SUMMARY.

Density stratification can result from vertical variations in the concentration of solutes or sediments or in temperature and it is important in many geophysical flows. Turbulence and vertical mixing are suppressed by density stratification. Mixing processes which may be important in determining the behaviour of saline pools include: entrainment associated with interfacial velocity shear; mixing associated with two-dimensional features of the flow as it enters and leaves a saline pool; and convective mixing associated with surface cooling.

Several mixing regimes which depend on the Richardson number exist in turbulent entrainment. Theoretical models of the processes associated with each of these regimes were briefly discussed. Two different entrainment relationships were then considered and an empirical relationship proposed by Christodoulou (1986) was selected as being typical of the entrainment observed in experimental studies reported in the literature. Modelling of density stratification was discussed briefly and it was argued that a two-layer model should be used to simulate saline pools.

CHAPTER 8 - THE BEHAVIOUR OF SALINE POOLS.

8.1 INTRODUCTION.

The occurrence of saline pools in the Wimmera River was first documented by Anderson and Morison (1989c; 1989g). These pools are characterised by decreasing salinity with elevation in the water column and are usually located in scour holes in the river. Typically there is also an oxycline and a thermocline associated with the halocline (Anderson and Morison, 1989c). The salinity and temperature changes over the water column lead to stable density profiles dominated by changes in salinity. Figure 8.1 shows a schematic of a saline pool.

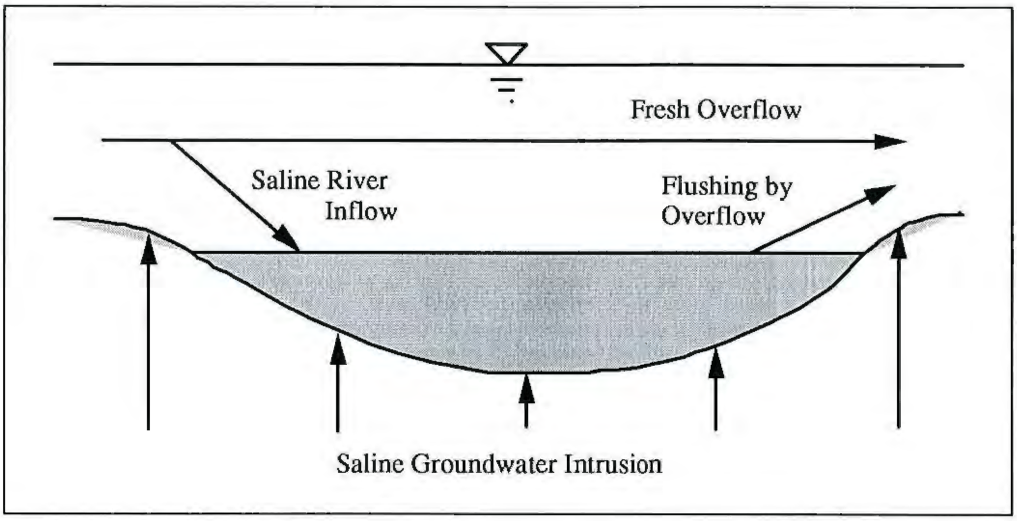


Figure 8.1: Schematic diagram of a saline pool.

This chapter briefly discusses previous work conducted on saline pools in the Wimmera River. A description of specific saline pool sites and data used in this study is then provided and the seasonal behaviour of saline pools is discussed. Flushing of saline pools is examined using both field observations and laboratory studies. Formation of saline pools due to groundwater intrusion directly into the stream channel and to buoyant inflows from upstream is discussed.

8.2 PREVIOUS WORK.

Anderson and Morison (1989c; 1989g) conducted a series of water quality surveys at various sites along the Wimmera River during 1986 and 1987. The first of these surveys was conducted in May, June and July of 1986 after an extensive period of low flow. Subsequent surveys were conducted on the 10/7/86 after a small event flow (estimated peak flow of 700 Ml/d at Dimboola on 9/7/86), between the 1/8/86 and the 5/8/86 after a flow event for which the peak mean daily discharge at Horsham was 3130 Ml/d on the 26/7/86, from the 23/2/87 to the 3/3/87 after the river had returned to low flows and between the 30/3/87 and 5/4/87 after a period of experimental environmental release. Based on these surveys Anderson and Morison (1989c; 1989g) make a number of qualitative observations on the behaviour of saline pools in the Wimmera River.

8.2.1 FORMATION OF SALINE POOLS.

Stratification develops during periods of low but continuous flow and the salinity and temperature of the saline water in individual stratified pools are usually similar for different episodes of stratification. This similarity along with similarities between the relative cationic concentrations (Na>>Mg>Ca) of the saline water in the pools and the adjacent Parilla Sands Aquifer is strongly indicative of a groundwater source for the saline water. In addition, seepage of groundwater into the stream through its banks was observed in many locations (Anderson and Morison, 1989c; 1989g).

Another mechanism for the formation of stratification was also proposed by Anderson and Morison (1989c; 1989g). This involves the inflow of saline river water from upstream which then collects in depressions in the stream bed. Evidence for this mechanism consists of observations of increases in depths of saline water layers and in the salinity at the stream bed after small flow events. Unfortunately Anderson and Morison used the river bed as the datum for their vertical profiles of salinity, temperature and dissolved oxygen. Thus measurements relied upon relocating the deepest point in each pool and on the invert level of the pool being constant. Uncertainty in the datum for vertical profiles makes comparison of the profiles difficult.

8.2.2 FLUSHING OF SALINE POOLS.

Anderson and Morison (1989c; 1989g) conducted surveys following two natural flow events in the Wimmera River. The first event, for which the peak flow was estimated to be 700 Ml/d at Dimboola, caused partial mixing of some stratified pools. The second event, which had a peak discharge of 3 130 Ml/d at Horsham, led to substantial or complete mixing of all the pools surveyed.

An experimental environmental release was made to the Wimmera River between the 25th of February 1987 and the 10th of April 1987. During the first seven days of this period a discharge of 100 Ml/d was released. This flow was subsequently reduced to 50 Ml/d. This flow improved the dissolved oxygen content of the upper part of the water column at saline pool sites but caused minimal mixing of saline pools themselves (Anderson and Morison, 1989c).

Anderson and Morison's results indicate that saline pools were not mixed significantly by a flow event with a peak discharge of 700 Ml/d, but they were completely mixed during an event with a peak discharge of 3130 Ml/d. However this does not imply that steady flows less than 700 Ml/d would not cause significant mixing. Events with a relatively small peak discharge also tend to have a short duration Therefore the limited mixing observed during the smaller event may have been a result of a short duration of high flow rather than the flow rate itself. The results from the experimental flow release indicate that steady flows of 50 Ml/d do not cause significant mixing.

8.3 FIELD MONITORING OF SALINE POOLS.

Several sets of data relating to saline pools in the Wimmera River have been used in these analyses. Saline pool sites used in this study were Lower Norton 1, Lower Norton 2, Big Bend, Polkemmet and Tarranyurk (Figure 8.2).

Two saline pools, Lower Norton 1 and Big Bend, have continuous salinity monitors installed. These are operated by the Rural Water Corporation of Victoria (RWC) and consist of two or three salinity sensors connected to a data logger and installed on a vertical cable. The location of these sensors within the water column is summarised in Figure 8.3. In addition to the continuous monitoring, the RWC has collected vertical salinity and temperature profiles at these two sites since October 1992. As part of this project vertical salinity and temperature profiles were closely monitored during two mixing events, one at Lower Norton 2 and the other at Tarranyurk. Profiles were also collected during low flow periods at all these sites except Polkemmet as part of this project. Vertical salinity profiles collected by Kaiela Fisheries Research Station (KFRS) at the five stations were also made available for analysis (T. Ryan, KFRS, Per. Com.).

For purposes of analysis, all the vertical profiles, except for those relating to mixing events, were combined. There are therefore three sets of data: manually measured vertical salinity profiles excluding mixing events, manually measured vertical salinity profiles from mixing events and continuously monitored salinity at specific locations within the water column at two sites.

In addition to the salinity data, cross-sections were collected at each site and used to construct maps of the bed topography (personnel from KFRS provided this information for Polkemmet). Surveys of the distribution of saline pools along the river were conducted in two different areas. Finally, stream discharge data were obtained from RWC gauging stations.

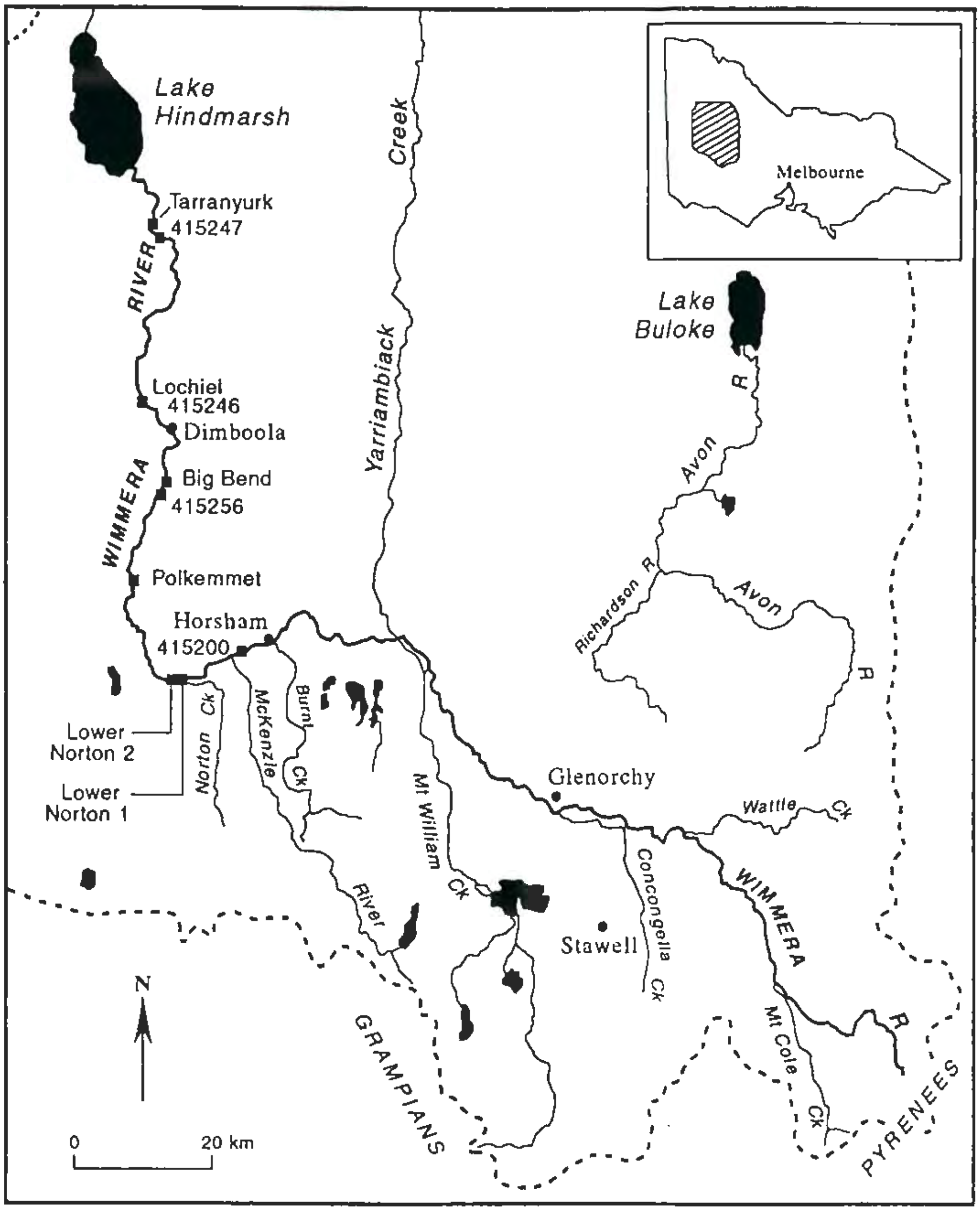


Figure 8.2: Location of saline pool monitoring sites in the Wimmera River.

8.4 DESCRIPTION OF SALINE POOL SITES.

Saline pools are commonly found along the Wimmera River downstream of Roseneath. They are typically located on channel bends since the denser, more saline water, collects in topographic low points which are usually located on channel bends. In many cases the saline pools are located in relatively deep and wide channel sections because it is these sections which retain significant amounts of almost still water during periods of low or zero flow. The five sites used in this study are described briefly below.

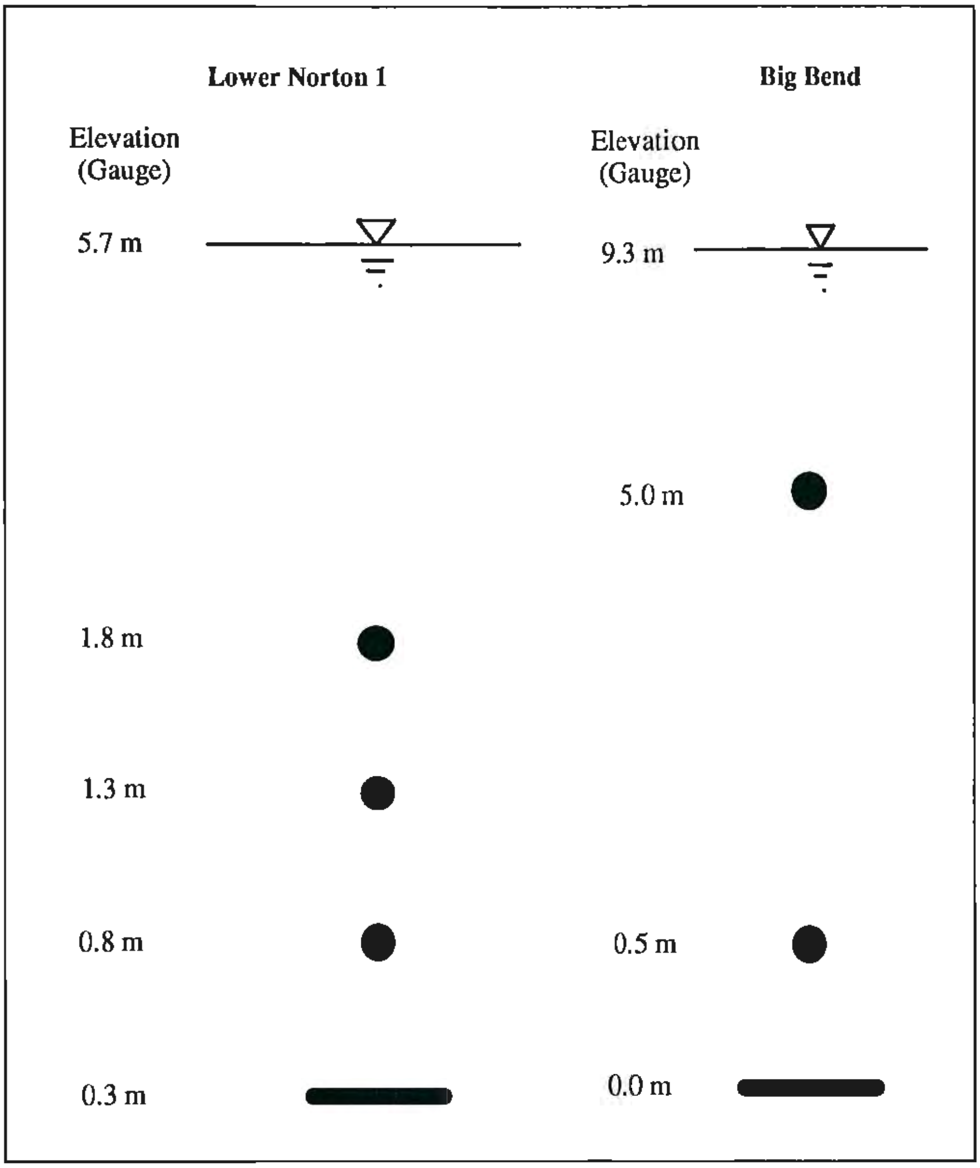


Figure 8.3: Location of continuously monitored salinity sensors at Lower Norton 1 and Big Bend.

Approximate pool dimensions, depths and discharges are provided to give the reader an appreciation of the characteristics of the individual sites and differences between the sites. Approximate discharges and associated depths are given for which most of the saline water has been flushed from a particular saline pool. These are depths and discharges observed during the first significant flow event after extensive periods of low flow. Since flow conditions at the time of complete mixing depend on the degree of stratification before the flow event and on the characteristics of the hydrograph these figures are indicative only. These issues are discussed further in Chapter 9.

Lower Norton 1 is a gentle bend which contains 5 m maximum depth of water during low flow periods in a stream channel that has an approximately constant

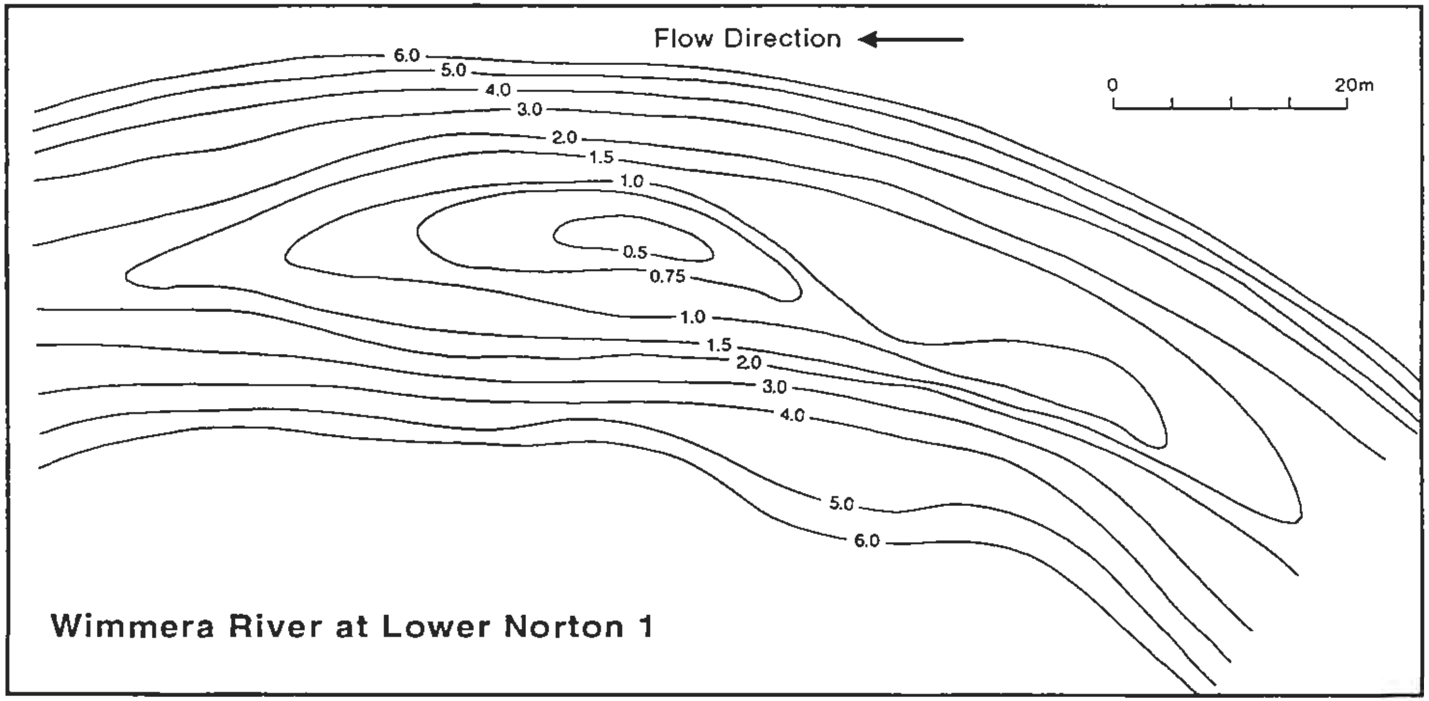


Figure 8.4: Channel Geometry at Lower Norton 1. The highest contour shown corresponds to the low flow water level.

Figure 8.5: View of Lower Norton 1 looking downstream.

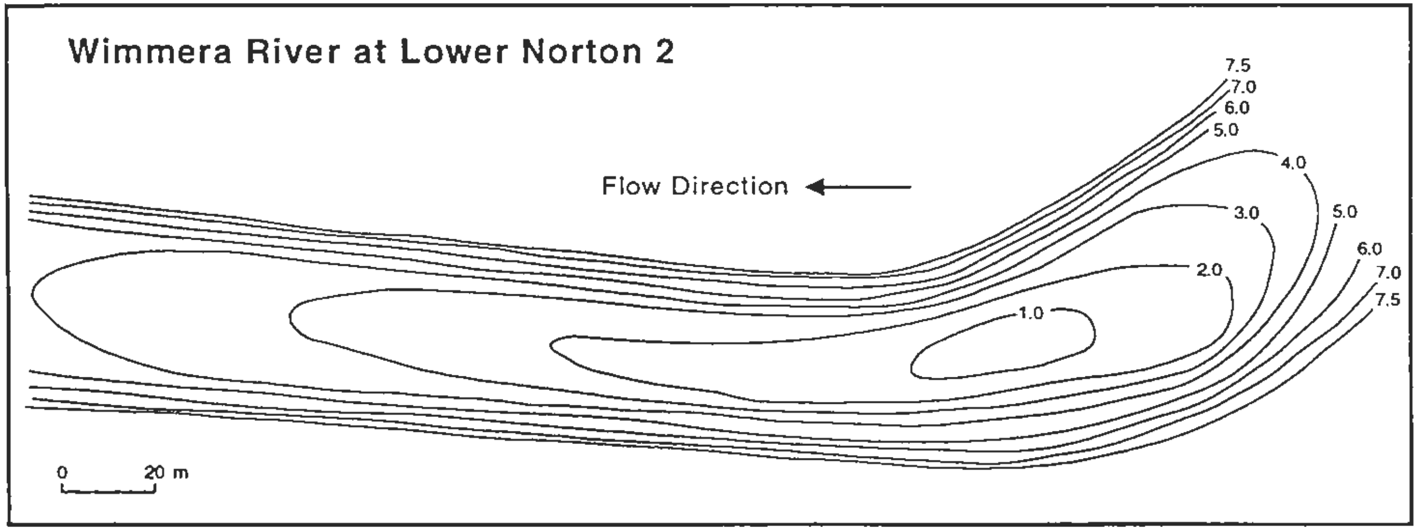


Figure 8.6: Channel Geometry at Lower Norton 2. The highest contour shown corresponds to the low flow water level.

Figure 8.7: View of Lower Norton 2 during a mixing event.

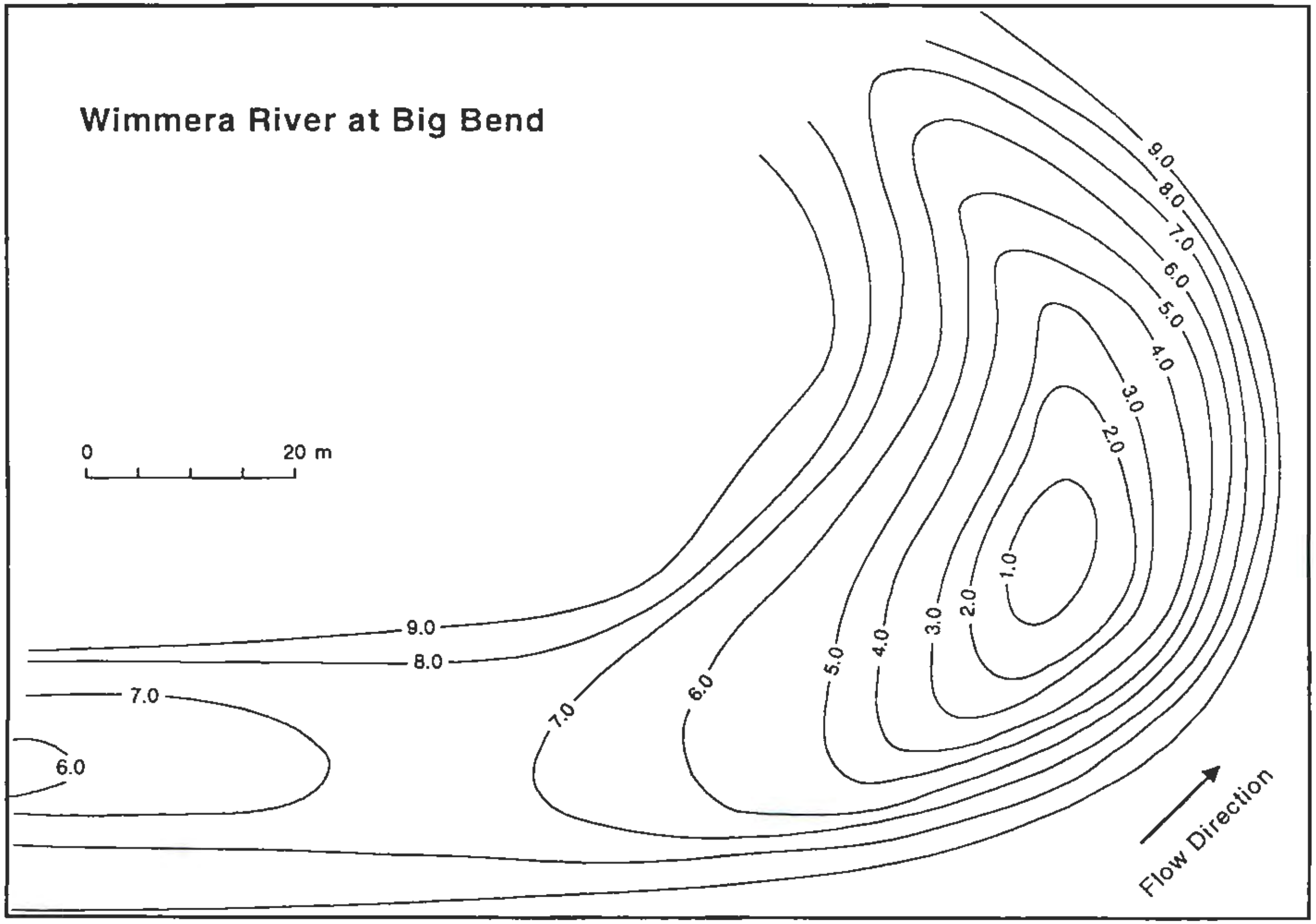


Figure 8.8: Channel Geometry at Big Bend. The highest contour shown corresponds to the low flow water level.

Figure 8.9: View of Big Bend looking upstream.

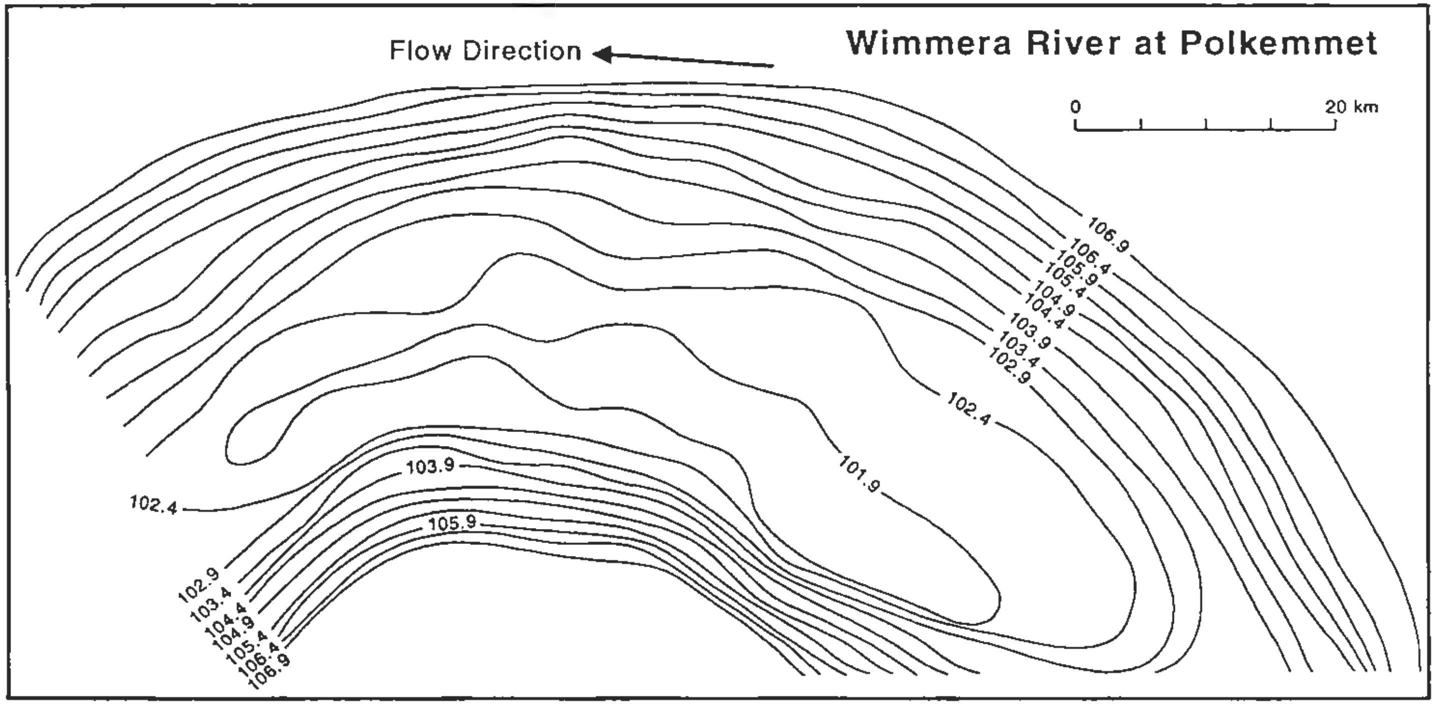


Figure 8.10: Channel Geometry at Polkemmet. The highest contour shown corresponds to the low flow water level.

Figure 8.11: View of Polkemmet looking downstream.

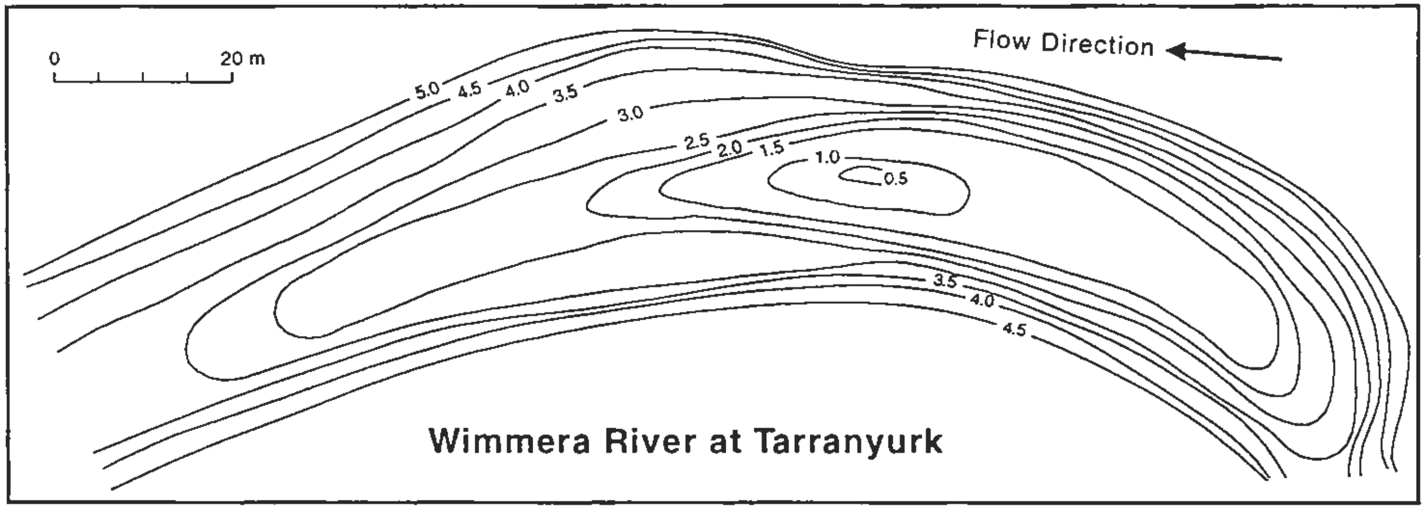


Figure 8.12: Channel Geometry at Tarranyurk. The highest contour shown corresponds to the low flow water level.

Figure 8.13: View of Tarranyurk looking upstream.

width of 35 m. The scour depression on this bend is approximately 1.5 m deep and has a volume of 1 150 m^3. Salinities up to 14 000 EC have been observed in the lower layer at this site. Most of the saline water has been mixed from this saline pool when the discharge has reached 1 000 Ml/d or when the depth is approximately 6.5 m. Figure 8.4 shows the topography of the river bed and Figure 8.5 is a view of this site looking downstream at this location.

At Lower Norton 2 the stream channel reduces in width from 60 m to 40 m as the flow moves around a gentle bend. The deepest point at this site is located near the sharpest part of the bend and the scour depression extends though the bend and for a distance of approximately 180 m downstream of the bend. This depression has a volume of 10 900 m^3. Figure 8.6 shows the bed topography at Lower Norton 2 and Figure 8.7 shows this site during a mixing event. The maximum water depth during low flow periods at this site is approximately 7 m and major mixing has occurred when the depth has reached 9 m or the discharge has reached 2 000 Ml/d.

Big Bend is a sharp bend through which the channel widens from approximately 30 m upstream of the bend to 60 m and then contracts to approximately 40 m at the exit of the bend. This pool contains 9 m depth of water during low flows and 10 m when the discharge is 1 500 Ml/d at which point major mixing has occurred. The scour depression at this site is 7 m deep and has a volume of 6 500 m^3. Figure 8.8 shows the bed topography at this site and Figure 8.9 shows Big Bend during a low flow period.

Polkemmet is a moderate bend that is 5.5 m deep during low flows and 45-50 m wide. When the discharge has reached 2 000 Ml/d the depth of the pool is 8.5 m and significant mixing has occurred. The scour depression is approximately 1.8 m deep and has a volume of 530 m^3. During periods of major stratification saline water in this depression can become linked to that in a second scour depression approximately 200 m downstream. The total depth of stratification then exceeds 1.8 m. Figures 8.10 and 8.11 show Polkemmet during low flow conditions and the bed topography.

Tarranyurk is a saline pool site on the Wimmera River downstream of Lochiel and is thus outside the section of stream included in the hydrodynamic model (Chapter 6); however it has been included in this analysis because a mixing event was monitored there. At the point where flow enters this bend there is a rapid expansion in channel width from 15 m to 30 m and a sudden change in flow direction. This is due to an extensive bar formation immediately upstream of the bend. The radius of curvature and width of the channel downstream of this entrance section are then approximately constant. The deepest point occurs approximately 60 m downstream of the bend entrance and a scour depression occupies the entire length of the bend. This depression is approximately 3 m deep and has a volume of 2 400 m^3. During low flows the maximum depth in the bend is 4.5 m which increases to 4.8 m at a discharge of 500 Ml/d at which time major mixing has occurred. Figure 8.12 provides a view of the site and Figure 8.13 shows the bed topography.

8.5 SEASONAL BEHAVIOUR OF SALINE POOLS.

This section presents a brief qualitative discussion of the behaviour of saline pools at five sites in the Wimmera River. Quantitative analyses are provided in subsequent sections.

8.5.1 LOWER NORTON 1 AND 2, POLKEMMET AND TARRANYURK.

Figure 8.14 shows the temporal variation in the volumes of water with salinities greater than 2 500, 5 000 and 10 000 EC stored in the scour depression at Lower Norton 1. Points on the graph are joined when continuous stratification existed between the two sample times; provided that this can be confirmed from continuously monitored data or can be inferred from the absence of discharge events with peak discharges exceeding 500 Ml/d. The discharge for the Wimmera River at Horsham, which is ~10 km upstream of Lower Norton 1, is also shown and times when vertical profiles indicated that the water column was unstratified are identified.

During periods of extended low flow there is continuous and extensive stratification at Lower Norton 1 (Figure 8.14). It is also clear from the large reduction in volume of saline water stored that high flow events lead to the saline water being flushed from the scour pool. Furthermore, it is apparent that stratification reappears under low but continuous flows. The stratification at Lower Norton 1 is a persistent feature which exists during periods of relatively low discharge. Given this behaviour and the extensive periods of low flow experienced in the Wimmera River, Lower Norton 1 would usually be stratified.

The behaviour of the saline pools at Lower Norton 2, Polkemmet and Tarranyurk is similar to that of Lower Norton 1 (Figures 8.15, 8.16 and 8.17). After mixing by high flow events, stratification becomes re-established as the discharge decreases to a low flow. The amount of saline water stored below the halocline increases with time after the return to low flow. This behaviour is typical of that

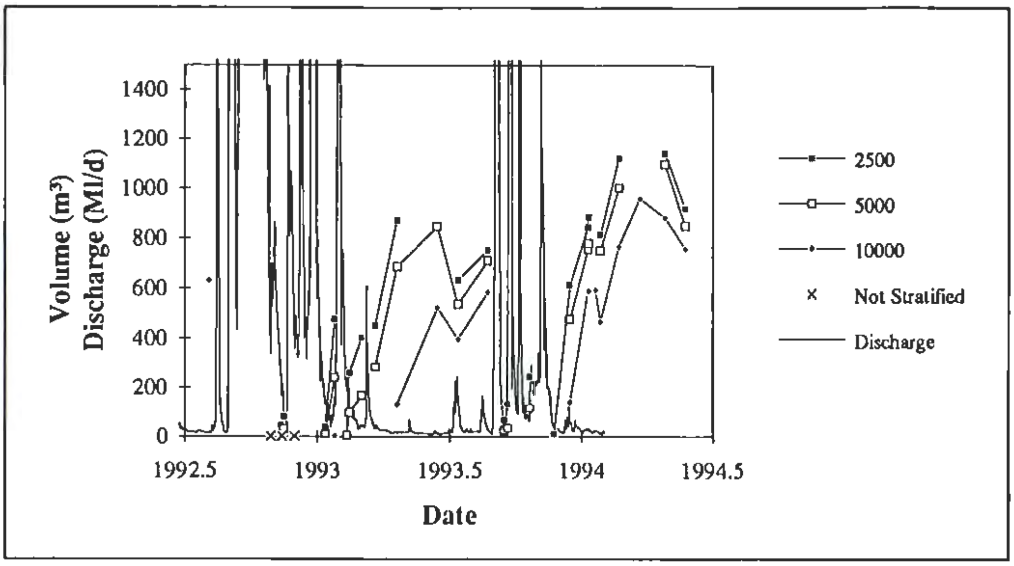


Figure 8.14: Volume of saline water exceeding 2 500, 5 000 and 10 000 EC units stored at Lower Norton 1. Stream discharge and occasions when the water column was not stratified are also shown.

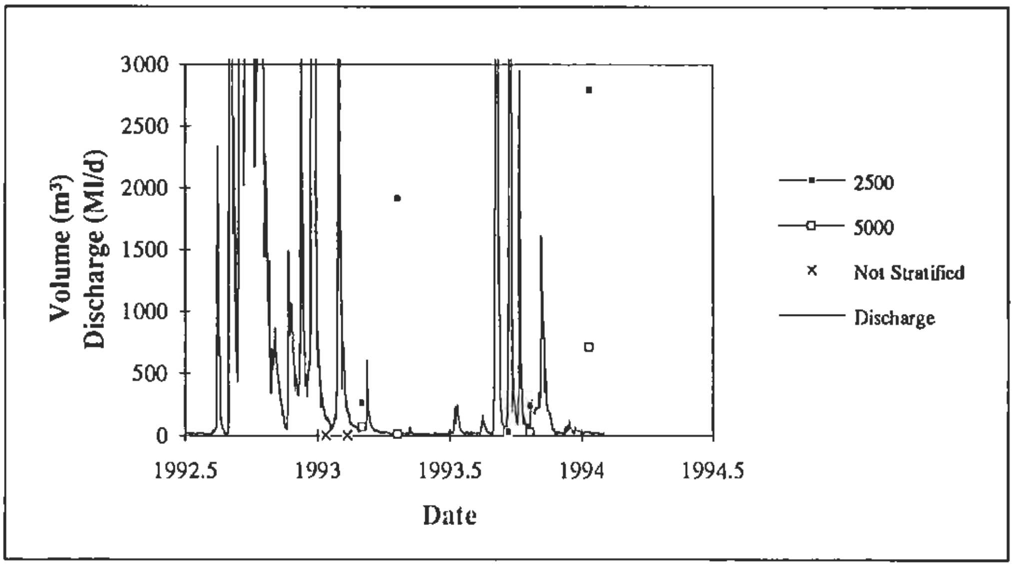


Figure 8.15: Volume of saline water exceeding 2 500, and 5 000 EC units stored at Lower Norton 2. Stream discharge and occasions when the water column was not stratified are also shown.

expected when the stratification results from saline groundwater seeping directly into the stream channel.

8.5.2 BIG BEND.

Figure 8.18 shows the volume of water with a salinity greater than 2 500, 5 000 and 10 000 EC stored in the scour depression, and the stream discharge at Big

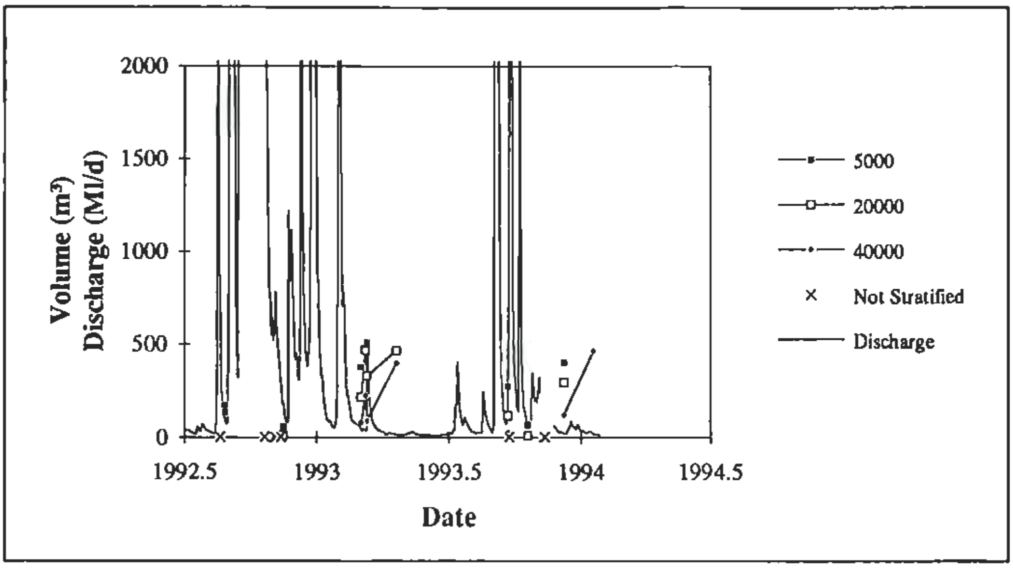


Figure 8.16: Volume of saline water exceeding 5 000, 20 000 and 40 000 EC units stored at Polkemmet. Stream discharge and occasions when the water column was not stratified are also shown.

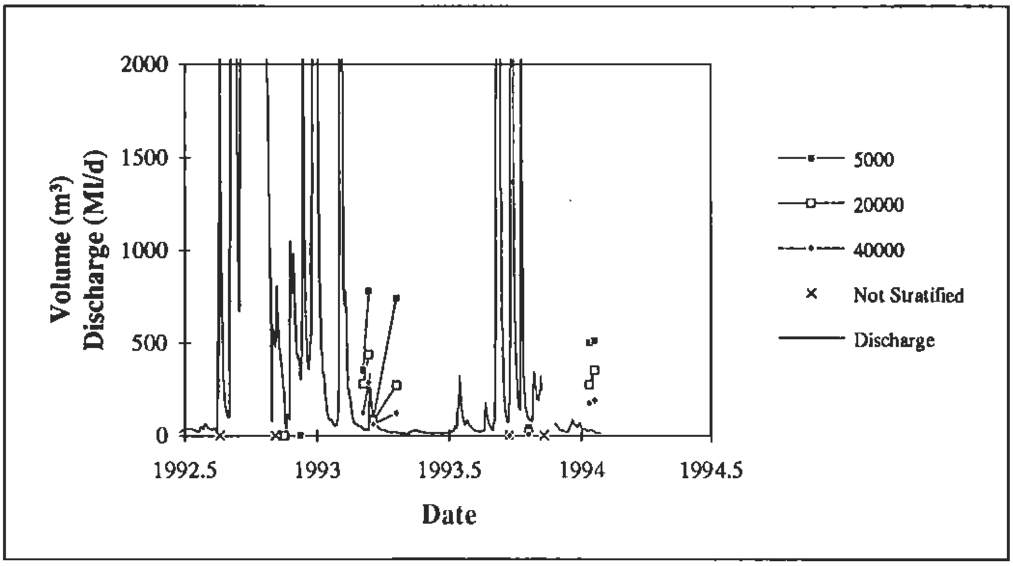


Figure 8.17: Volume of saline water exceeding 5 000, 20 000 and 40 000 EC units stored at Tarranyurk. Stream discharge and occasions when the water column was not stratified are also shown.

Bend. The behaviour of the saline pool at Big Bend is markedly different from Lower Norton 1. It is evident from the lack of stratification during the low flow period in the Autumn (Southern Hemisphere) of 1993 that the existence of stratification is not simply related to low flows. The difference in temporal behaviour of the saline pool at Big Bend when compared with the other sites is due

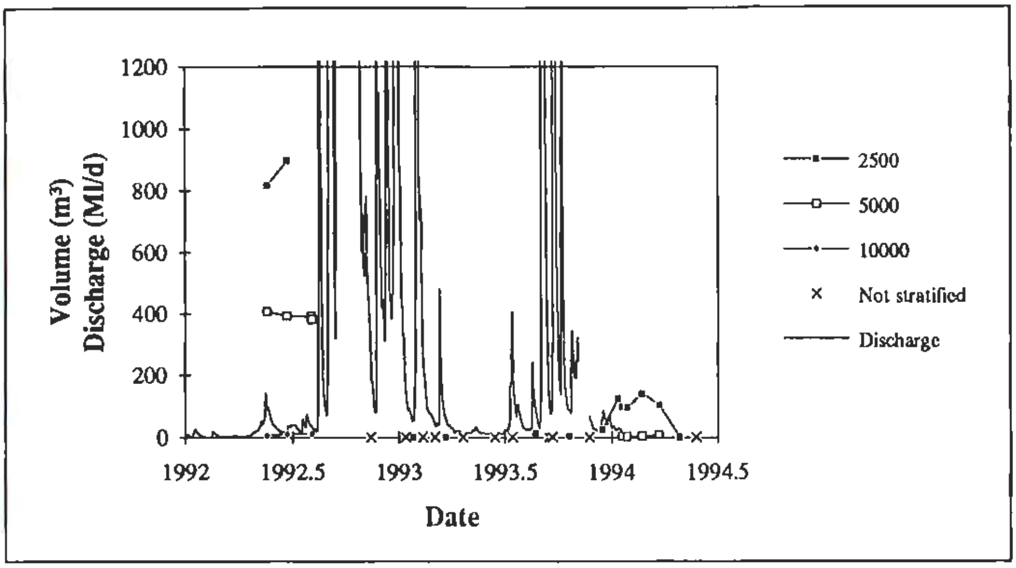


Figure 8.18: Volume of saline water exceeding 2 500, 5 000 and 10 000 EC units stored at Big Bend. Stream discharge and occasions when the water column was not stratified are also shown.

to saline river flows as well as saline groundwater flows contributing to the stratification and to mixing by surface cooling as well as river flows. These processes are discussed further below.

8.6 SPATIAL DISTRIBUTION OF SALINE POOLS.

Longitudinal surveys conducted in the Lower Norton and Polkemmet localities (Figure 8.2) on the 11/1/94 and 12/1/94 respectively show that saline pools are not found in every scour pool along a stream reach. This fact was also noted by Anderson and Morison (Anderson and Morison, 1989c; 1989g). River flows at Horsham were less than 100 Ml/d and generally less than 50 Ml/d for 50 days prior to the survey with the exception of 12 hours on the 14-15/12/93 when the discharge peaked at 120 Ml/d. Discharge at Horsham for the three months preceding these surveys are shown in Figure 8.19. The river at Lower Norton and Polkemmet consists of a large pool of permanent water generally in excess of 2 m deep with deeper scour holes found on channel bends.

The longitudinal surveys of saline pools were conducted by boat. Depths were measured in 0.3 m increments with a depth sounder and were noted when a significant depth change was observed. Surface and bottom salinities were measured in all scour depressions and, if there was any stratification, the depth of the halocline was determined. The location of the boat was determined using 1:25 000 scale aerial photographs.

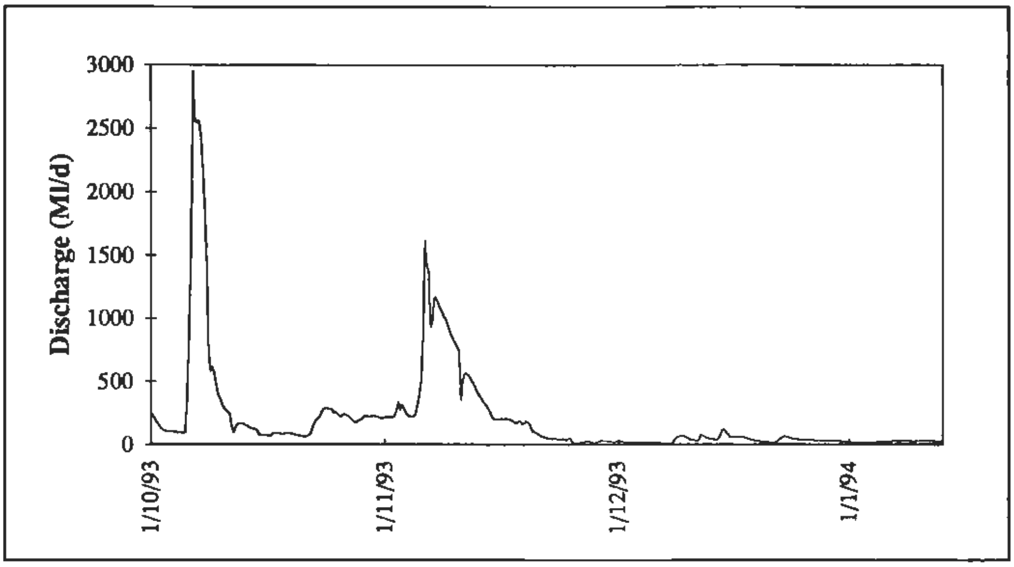


Figure 8.19: Discharge for the Wimmera River at Horsham between 1/10/93 and 12/1/94.

Figures 8.20 and 8.21 show the depth of the channel thalweg from the water surface and the location of haloclines. Approximately half the scour depressions at each locality were stratified. In the Lower Norton area approximately 25% of the river length was stratified. This would be expected to increase to some extent as scour depressions filled with saline water over the remainder of the low flow season (~150 days). At Polkemmet the proportion of the river length affected by stratification is approximately 18%; slightly less than at Lower Norton. This is due in part to an unusually long, straight and uniform section of channel between 1 900 m and 2 700 m.

During their study Anderson and Morison (1989c) measured a total of 160 vertical salinity profiles at 61 different sites along the Wimmera River between Horsham Weir and Lake Hindmarsh over 5 separate surveys. These data provide another estimate of the frequency of saline pools in the Wimmera River. If an increase of more than 1000 EC between the top and bottom salinity is used as an indicator of stratification (Anderson and Morison, 1989c), 38% of the measured profiles showed that the water column was stratified. Four of these surveys included 20 or more sites and on those occasions between 32% and 44% of the sites were stratified. The sites at which vertical profiles were measured varied to some extent between surveys and the number of profiles measured at a site varied between 1 and 5. Of the 61 sites, 57% were never stratified, 25% were stratified on some occasions and 18% were stratified on all occasions for which profiles were measured.

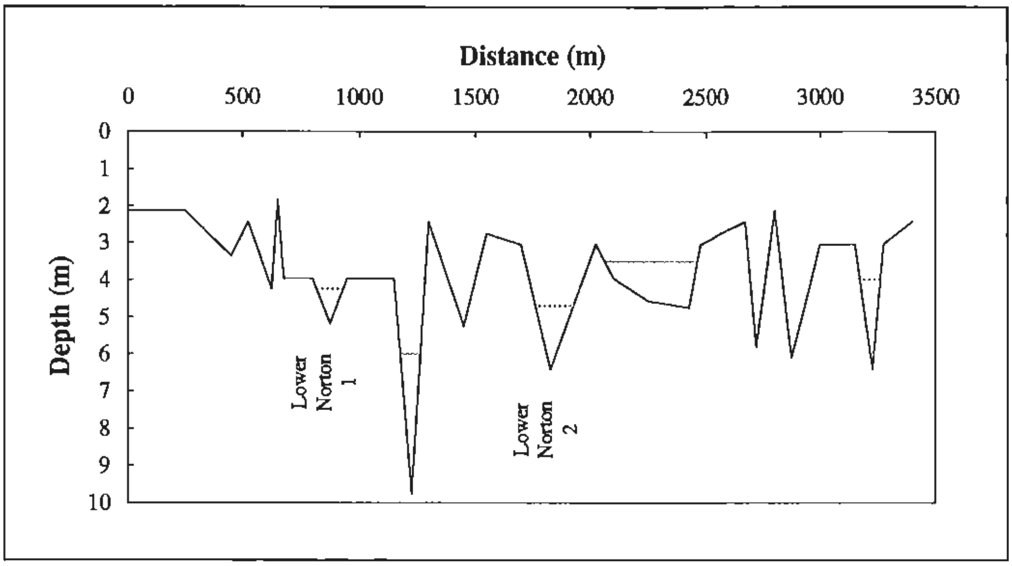


Figure 8.20: Longitudinal channel profile at Lower Norton. Haloclines are shown with dotted lines and the distance is meters downstream of the confluence of Norton Creek and the Wimmera River.

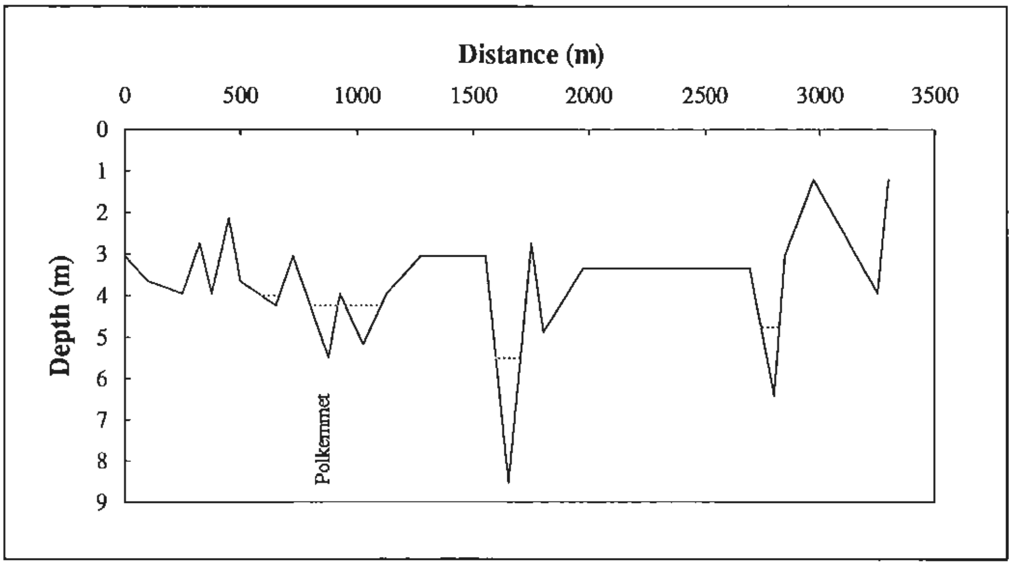


Figure 8.21: Longitudinal channel profile at Polkemmet. Haloclines are shown with dotted lines and the distance is meters downstream of the Polkemmet Road Bridge over the Wimmera River.

Anderson and Morison's (1989c) survey sites were selected primarily for a fish habitat assessment and were selected in a systematic way to ensure coverage of different habitat types (Anderson and Morison, 1989b). The sites are therefore not randomly located and may not provide a representative estimate of the proportion of sites which become stratified. Furthermore, while salinity profiles were always measured at the deepest point in the channel, it is not clear that this was always a scour hole. If some sites were not located within scour depressions, a bias leading to an underestimation of the proportion of scour holes that become stratified would exist. Given the different site selection methodologies, Anderson and Morison's (1989c) data are not directly comparable with the longitudinal surveys described above. However both surveys indicate that, while density stratification is a common phenomenon along the Wimmera River, there are a significant number of scour holes which do not commonly become stratified.

8.7 MIXING OF SALINE POOLS BY FRESH OVERFLOWS.

The role of fresh overflows in mixing of saline pools was investigated in both the field and the laboratory. An initial investigation of mixing in the field indicated that typical 1-Dimensional entrainment relationships presented in the literature (eg. Christodoulou, 1986) could not explain the observed mixing. This prompted the development of a laboratory program which was carried out by Nolan (1994) and aimed to provide additional information relating to the processes and rates of mixing in saline pools.

8.7.1 FIELD INVESTIGATIONS.

Sequences of vertical salinity profiles were collected at Lower Norton 2 and Tarranyurk during flow events. The data from Lower Norton 2 were collected prior to the laboratory experiments and motivated those experiments while those collected at Tarranyurk complement the Lower Norton 2 data. A preliminary analysis of the Lower Norton 2 data is described in this section.

Detailed vertical conductivity profiles were collected for an entrainment event on the 15-16/8/92 at Lower Norton 2. Over a period of 25 hours 2.5 m of mixing was observed. Figure 8.22 presents conductivity profiles measured at the beginning, in the middle and at the end of the period. The vertical position of the conductivity probe could be determined with ±0.025m. Field observations of conductivity in the interfacial region (the region of rapid conductivity change) showed significant variation and the range of these variations was noted. It is likely that this variation was due to interfacial waves passing the sensor. The actual conductivity was determined by taking the midpoint of the range for these points except when a predominant conductivity was noted during data collection.

River stage was also monitored throughout this period and was used to determine the discharge (Figure 8.23). The rating curve used to calculate discharge at this site was developed from several stream gaugings some of which were performed during field trips and others of which were performed by the RWC during 1992.

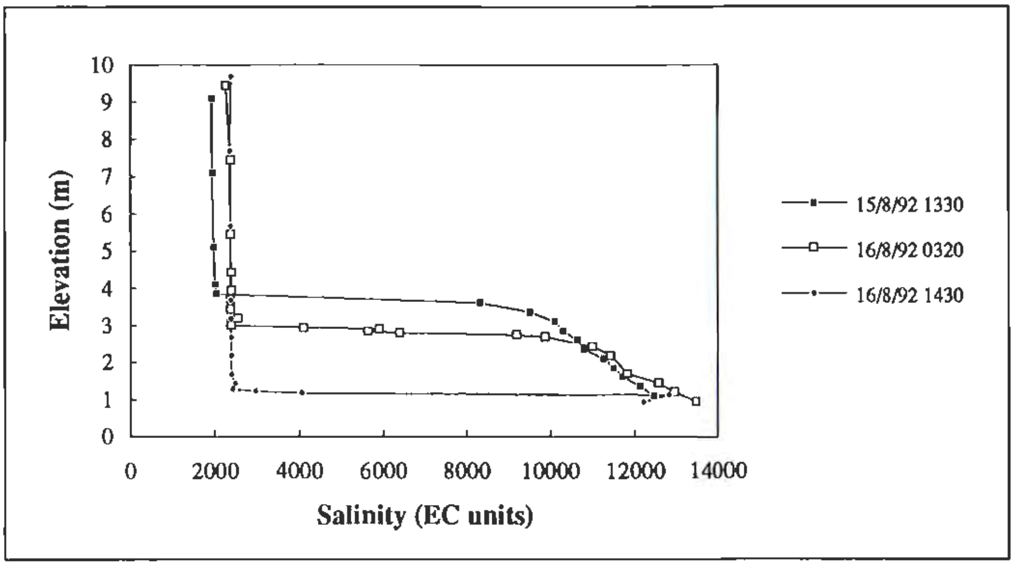


Figure 8.22: Salinity profiles collected at Lower Norton 2 during a mixing event on the 15th and 16th of August 1992.

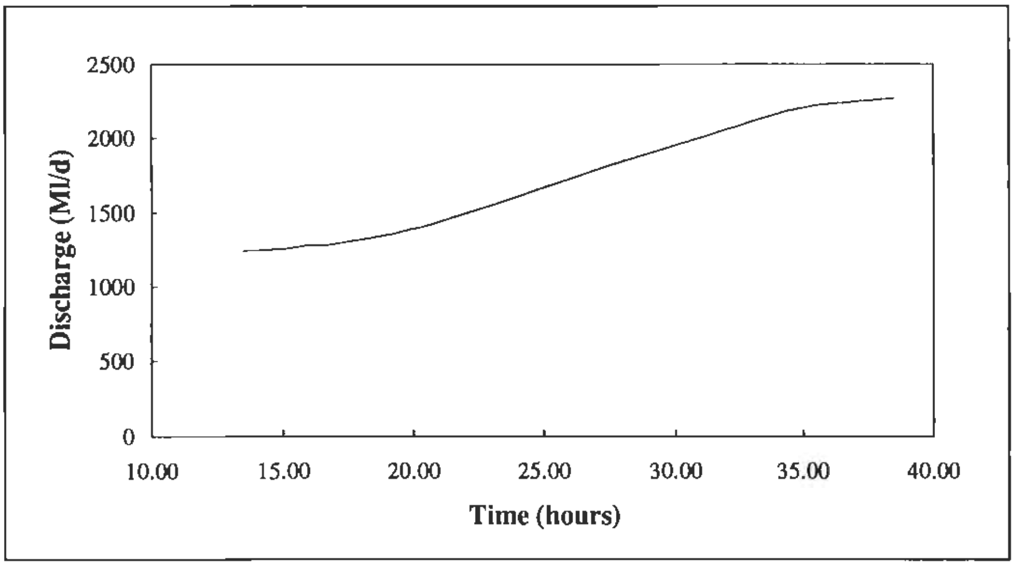


Figure 8.23: Discharge during mixing event on the 15th and 16th of August 1992 at Lower Norton 2. Time is hours since 0000 hours on the 15/8/92.

From these profiles the position of the interface was determined by finding the interval with the maximum vertical conductivity gradient and averaging the elevations at either end of this increment. The elevation of the interface is subject to errors of approximately ±0.05m. A second-order polynomial was fitted to the interface data (Figure 8.24) and this can then be differentiated to provide the velocity, u_e, at which the interface moved down due to mixing.

A bulk Richardson number, Ri_b, was calculated using:

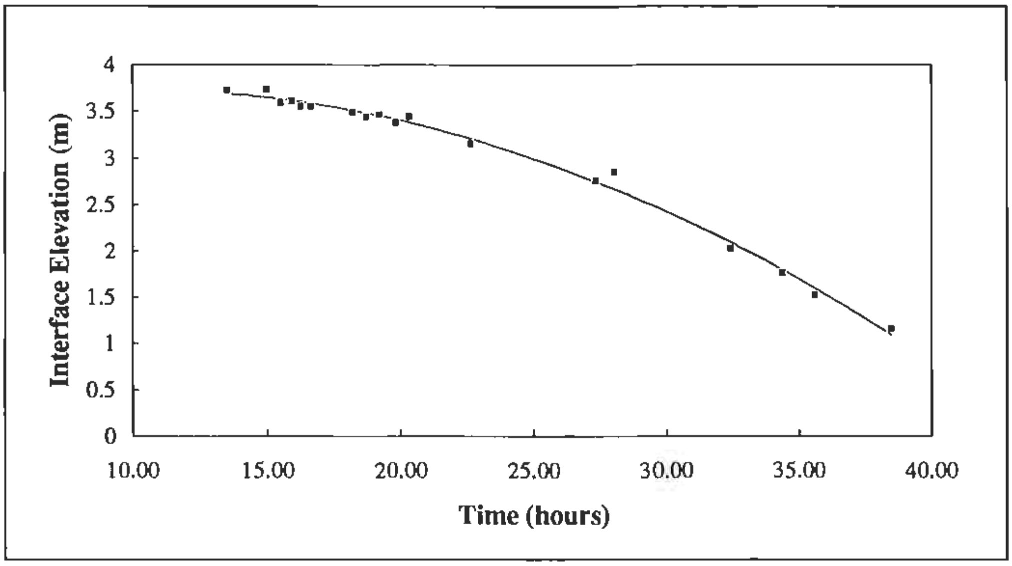


Figure 8.24: Variation in interface elevation during mixing event on the 15th and 16th of August 1992 at Lower Norton 2. Time is hours since 0000 hours on the 15/8/92.

$$Ri_b = \frac{g' h_1}{U_1^2} , \tag{8.1}$$

in which g' is the buoyancy, h_1 is the depth of the mixed layer and U_1^2 is the cross-sectional mean velocity of the mixed layer at the deepest point of the scour depression. The density difference was calculated as 0.6*0.0007*ΔEC, where ΔEC is the difference in electrical conductivity over the interface, 0.6 is an empirical factor which converts from EC units to parts per million and 0.0007 converts from ppm to kg/m^3. ΔEC was estimated by determining data points at the limits of the interface from plots of the vertical conductivity profile and then taking the difference between the conductivities at the two points. Figure 8.25 shows the variation in Ri_b during the event. The entrainment parameter, $E = U_e/U_1$, is also shown.

8.7.1.1 Mixing Relationships.

The observed values of E are significantly greater than those expected based on relationships between E and Ri reported in the literature. Typical values of E reported in the literature can be calculated using the relationship Christodoulou (1986) developed by examining 12 significant experimental studies of entrainment. The values observed at Lower Norton 2 are between 4 and 40 times larger than those calculated using Christodoulou's relationship.

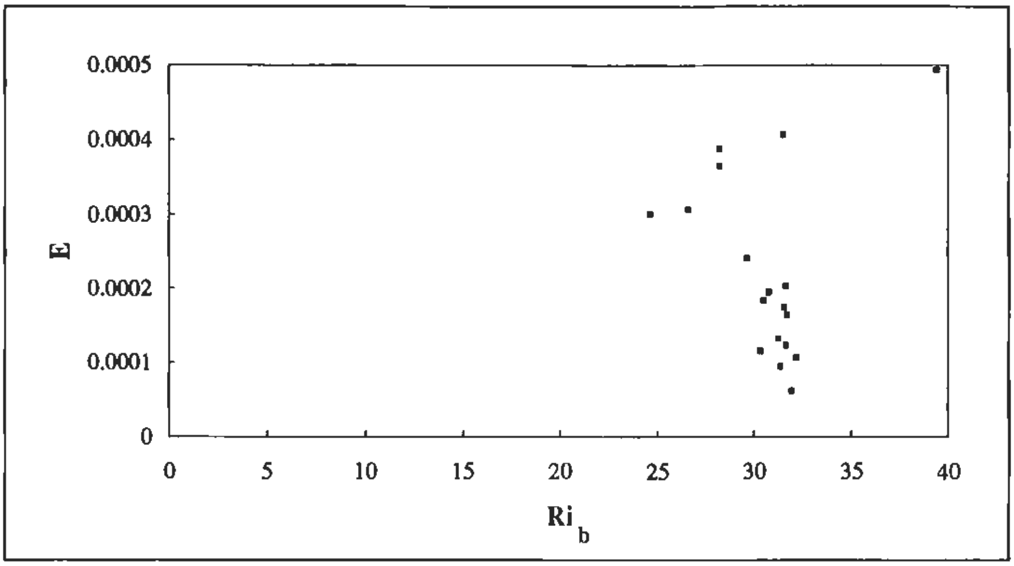


Figure 8.25: Relationship between E and Ri_b during the mixing event at Lower Norton 2 on the 15th and 16th of August 1992.

Figure 8.25 shows a plot of E vs Ri_b for Lower Norton 2. There is apparently no relationship between the two; however the literature dealing with the relationship between E and Ri_b nearly always reports a relationship characterised by E increasing as Ri_b is decreased. The relationship is usually of the form:

$$E = C_E \, Ri_b^n, \tag{8.2}$$

where C_E and n are positive and negative constants respectively. These relationships are typically based on well controlled laboratory experiments and the consistency of the reported form of the relationship gives confidence in that form. Thus the data from Lower Norton 2 indicate that the simple 1-Dimensional entrainment relationships reported in the literature are unlikely to be applicable to the mixing of saline pools. The reasons for this result and the processes responsible for the observed behaviour were then investigated in the Laboratory.

8.7.1.2 Direction of salinity transport.

Vertical mixing in stratified fluids can occur in either one or both directions. A basic requirement for transport into a layer is that that layer be turbulent (see §7.3.1.1). Because saline pools consist of a moving upper layer and a horizontally confined lower layer which would not be turbulent, it would be expected that transport would only occur from the lower to the upper layer. Profiles from Lower Norton 2 (Figure 8.22) show that the electrical conductivity profile in the lower layer is unaffected by mixing until the halocline passes. This indicates that mixing occurs by the upper layer progressively eroding the lower layer and that transport only occurs from the lower to the upper layer as expected.

8.7.2 LABORATORY INVESTIGATIONS.

8.7.2.1 Previous Studies.

Armfield and Debler (1993) conducted a series of laboratory investigations of the flushing of a square cavity initially filled with saline water by a fresh over flow. These experiments were discussed in detail in §7.3.2. Armfield and Debler's experiments were dominated by phenomena that resulted from the flow being started rapidly and by a flow separation which occurred at the cavity entrance. It was argued (see §7.3.2) that neither of these features are likely to be significant during the flushing of saline pools. A series of laboratory experiments designed to overcome the above shortcomings was performed.

8.7.2.2 Experimental Configuration and Qualitative Observations.

Laboratory investigations were performed to fulfil two primary objectives: to clarify the processes of flushing of saline pools by a fresh overflow and to determine a relationship between bulk flow properties, such as the upper layer velocity and the rate of mixing. While the experiments were an integral part of this project, they were actually performed by Nolan (1994).

These experiments consisted of a two-layer stratified flow with a saline lower layer and a fresh upper layer and were conducted in a laboratory flume. The laboratory model (Figure 8.26) was based on the geometry of an idealised saline pool and the velocity range was based on Froude scaling. Saline water was fed into the bottom of this depression through a diffuser at a steady rate, which was set so that the interface remained within the depression, and a steady flow of freshwater was passed over the top of the saline layer. The experiment was then allowed to come to equilibrium at which time a sharp interface between the fresh and saline water existed and the depression was partially full of saline water. The water surface elevation was controlled with a fixed crest weir downstream of the depression so that it was approximately 140 mm above the top of the depression. The saline water was dyed with Rhodamine-WT to enable flow visualisation.

The predominant flushing mechanism observed during these experiments consisted of a thin layer of saline water flowing up the downstream slope of the depression and along the bed of the flume. The depth of this layer was of the order of $1/100^{th}$ of the depth of the upper layer. The interface along the majority of the depression was approximately horizontal; only beginning to slope significantly at the downstream extremity of the main stratification or at the point where the shallow flow up the downstream slope of the depression began (Figure 8.26). Sampling showed that the saline water flowing from the cavity was a mixture of upper and lower layer water.

Interfacial waves were observed to form near the upstream extremity of the interface. These waves grew as they progressed along the interface and entrainment due to the breaking of these waves was observed during the experiments. This entrainment increased as the waves became larger further downstream and as the upper layer velocity increased or the density difference decreased. The flow up the downstream slope of the depression was continuous and, although it was steady at a sufficiently long time-scale, surges resulting from interfacial waves arriving at and progressing up the downstream slope were observed on some occasions. It is possible that the partially mixed water flowing from the downstream end of the depression results from partial mixing during wave breaking episodes.

It is known from Armfield and Debler's (1993) experiments that the flushing regime of a square cavity is significantly different to that observed in Nolan's (1994) experiments. One fundamental feature of the geometry of the depression is the slope of the entrance and exit to the depression. A series of experiments were conducted to clarify the range of entrance and exit slopes for which a continuous downstream outflow is the predominant flushing mechanism. These experiments were conducted with a depression that was 70 mm deep at the mid point and 1600 mm long at the top of the depression. End-wall slopes of 90°, 45°, 22°, 11° and 5° were used in these experiments. All these depressions, except for that with 5° end-wall slopes, had a flat section in the middle of the depression (Figure 8.27). Qualitative observations of the flow and quantitative measurements of mixing rates were conducted at two flow rates for each depression. The buoyancy (g') was fixed at 0.035 m/s^2.

When $\theta = 90°$ and $\theta = 45°$ the interface near the upstream extremity of the stratification dipped down. Further downstream the interface began to slope up and at the downstream extremity of the stratification it intersected the flume bed. The minimum interface elevation occurred approximately 1/4 of the way along the depression from the upstream end (Figure 8.27). A significant difference between the cavity aspect ratio (top length:depth) of Armfield and Debler's (1993) experiments (1:1) and the above experiments (23:1) exists.

On the upstream section of the interface waves were observed to travel upstream and mixing was observed in the region of the upstream end wall. A vortex located in the upper corner of the cavity is inferred from the interface slope and the

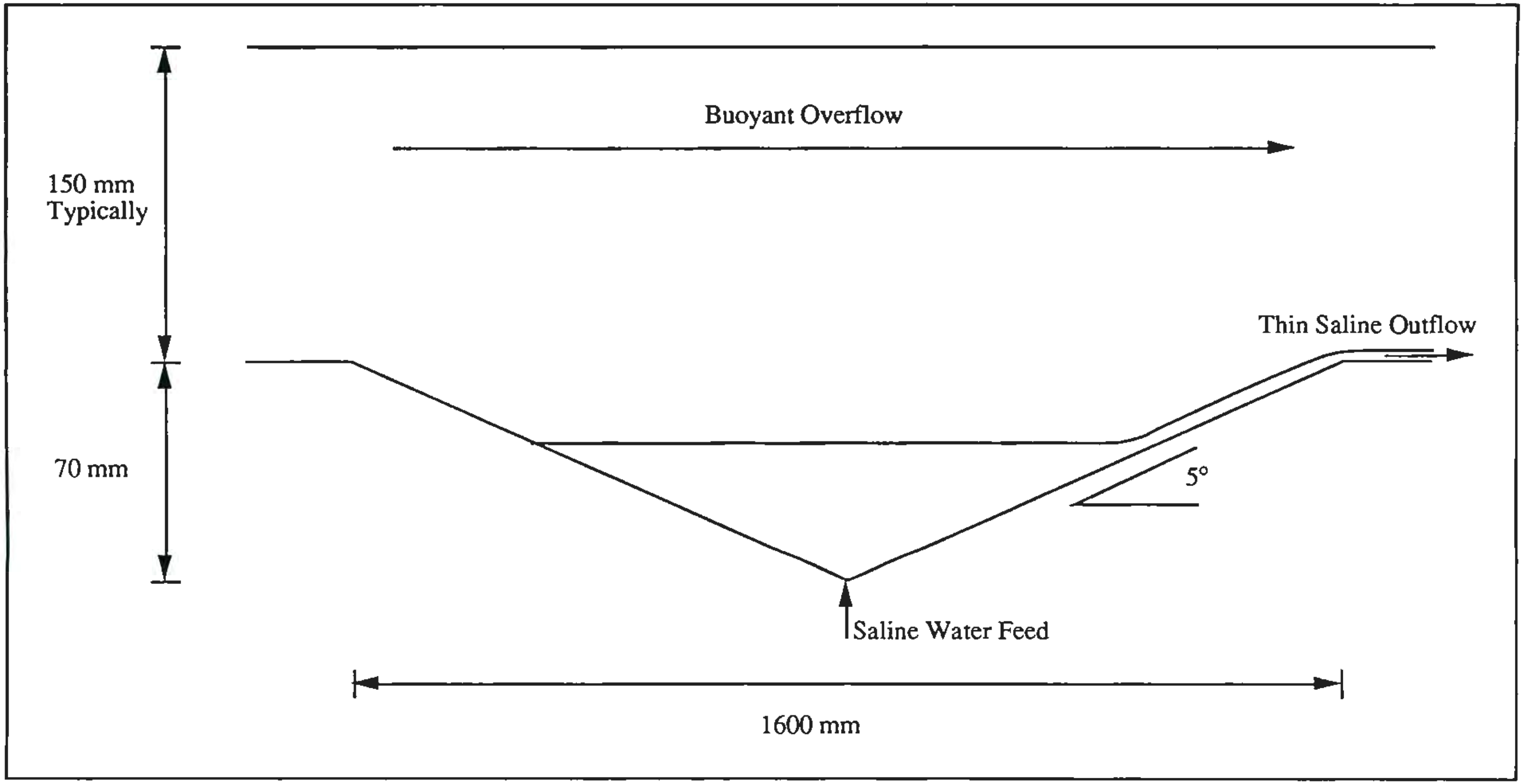


Figure 8.26: Flow geometry for laboratory experiments with end-wall slopes of 5°.

interfacial waves travelling upstream in this part of the flow. It is likely that this vortex is associated with a flow separation at or near the cavity entrance. Further downstream, significant waves were observed on the interface and these waves contributed to the flushing of the depression due to breaking and due to the caps of the waves passing out the downstream end of the depression whenever the top of the wave was higher than the top of the depression. Any flow up the downstream end of the depression was intermittent and strongly associated with the arrival of interfacial waves.

When the slopes of the end-walls of the depression are reduced to 22°, a continuous outflow up the downstream slope is present and the dip in the interface and mixing at the upstream wall are slight. For the depressions with 5° and 11° slopes the flows were similar to the earlier experiments. It can therefore be surmised that the flushing of the depression is dominated by a continuous flow up the downstream slope of the depression for angles less than approximately 20°. Given that bed slopes in the entrance and exit of scour depressions in the Wimmera River rarely exceed 10°, subsequent discussions will be restricted to this regime.

8.7.2.3 Parameterisation of Flushing.

Provided the slopes of the depression entrance and exit are less that 20°, the mechanism responsible for the majority of flushing involves a thin flow up the downstream end of the depression. The top of this layer is approximately parallel to the flume bed and its density is intermediate to the upper and lower layers. While the out-flowing layer is continuous, surges associated with the arrival of interfacial waves at the downstream slope can be observed.

The fluid flowing up the downstream slope is subject to a retarding buoyancy force along the slope arising from the slope of the interface and characterised by $g' \tan\theta$, where θ is the angle of the downstream slope to the horizontal. Velocity shear resulting from the overflowing layer must contribute some of the momentum required to lift the denser fluid up the downstream slope of the cavity. The surges observed in the outflow indicate that interfacial waves may also make a significant contribution to the outflow. At the point where the outflow begins, the shear is transmitted to the lower layer through an internal boundary layer that has developed over the length of the interface L (Figure 8.26). The interfacial wave train has also developed over the distance L. The interfacial length is therefore a significant length-scale from the point of view of momentum supply to the outflowing saline fluid. The velocity difference across the interface at the point

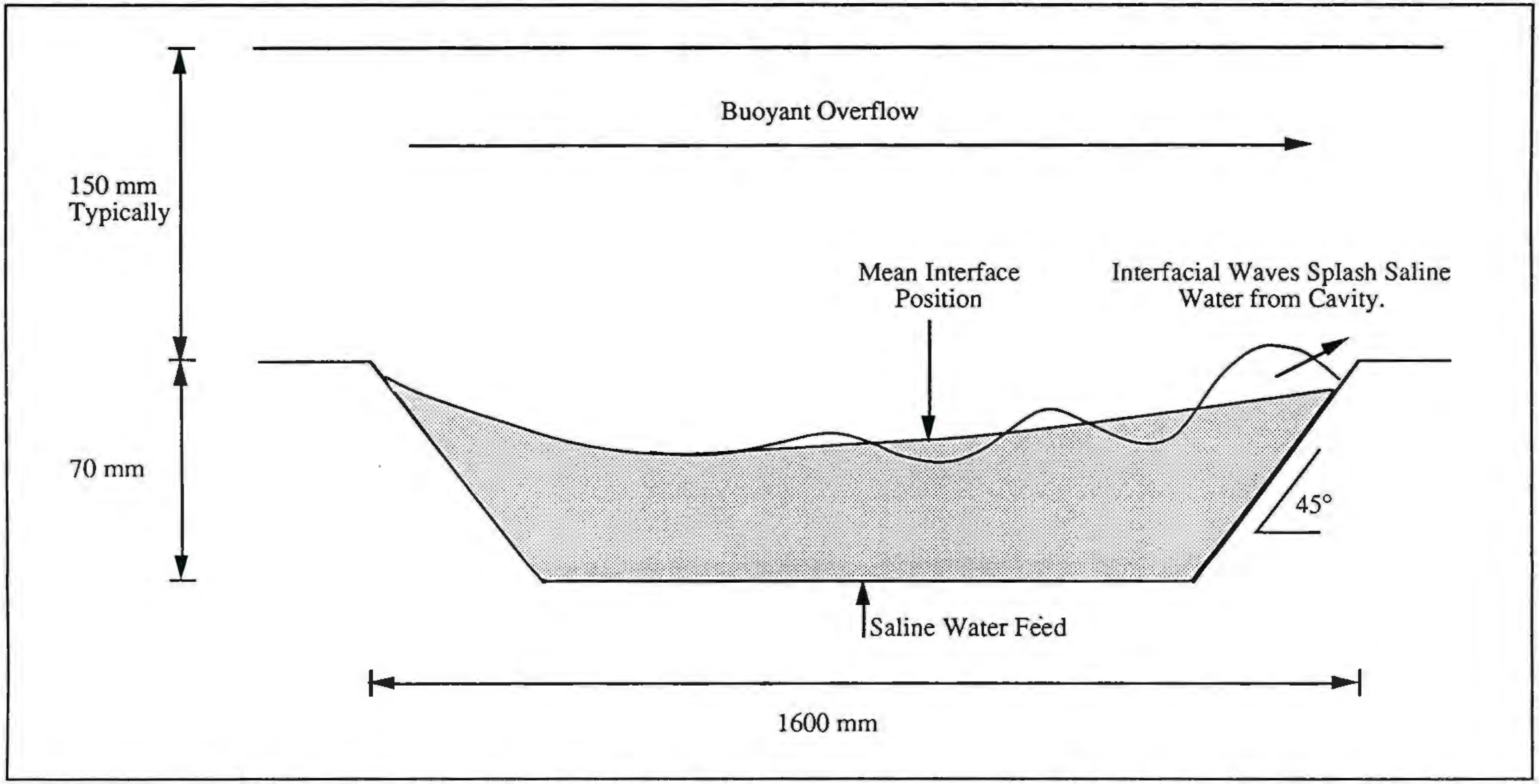


Figure 8.27: Flow geometry for laboratory experiments with end-wall slopes of 45°.

where the downstream outflow begins, U_{1D}, which, given the confined nature of the lower layer, can be approximated by the mean upper layer velocity U_1 is an appropriate velocity-scale. These scales combine to form a Richardson Number:

$$Ri_{Ls} = \frac{g' \tan\theta \, L}{U_{D1}^2} , \qquad (8.3)$$

Ri_{Ls} can be interpreted as the ratio of the buoyancy and inertia forces on the downstream slope. As Ri_{Ls} increases the buoyancy forces become larger relative to the shear forces and mixing would be expected to decrease.

Interfacial waves may influence flushing in two other ways. Firstly a small amount of mixing was observed due to turbulent entrainment associated with wave breaking episodes. Other length-scales may be important in this process. Secondly, the water in the downstream outflow may have resulted from partial mixing associated with wave breaking episodes.

8.7.2.3 Observed Mixing Rates.

Thirty-two experimental runs were conducted for buoyancies (g') of 0.014 m/s^2, 0.035 m/s^2, 0.070 m/s^2, 0.140 m/s^2, 0.209 m/s^2, 0.277 m/s^2 and 0.345 m/s^2 and a variety of upper layer velocities. The discharge from the flume, Q, and the rate at which the saline water was replenished were measured. Given that the experiments were conducted at a steady state, the saline water inflow rate was equal to the rate of flushing, Q_f, due to both the downstream outflow from the depression and turbulent entrainment. The water surface elevation, H, and elevation of the interface, H_i, were both measured. Unfortunately it was not possible to measure the velocity or thickness of the downstream outflow with the available equipment.

The upper layer velocity was calculated as $U_1 = \frac{Q}{B_i\,(H-H_i)}$, where B_i is the width of the interface, in this case equal to the width of the flume. L was calculated from the flume geometry by assuming that the interface was horizontal and the density difference was calculated using the salinity of the upper and lower layers. Ri_{Ls} was then calculated using the appropriate value of θ. A flushing velocity $U_f = \frac{Q_f}{B_i L}$ was calculated and normalised by U_1 to give a flushing parameter $F = \frac{U_f}{U_1} = \frac{Q_f}{U_1 B_i L}$. U_f is analogous to an entrainment velocity and F is analogous to the entrainment parameter.

A problem with using F to characterise the flushing is that it is not directly related to the dominant mixing mechanism which is the downstream outflow. A better characterisation of the flushing would consider the velocity and depth of the downstream outflow. Another problem is that the use of L in both the independent and dependent parameters has the potential to introduce a spurious correlation. For this reason the experimental results are first shown in a dimensional form from which the length-scale has been omitted. The dependent and independent variables are $\frac{Q_f}{U_1 B_i}$ and $\frac{U_1^2}{g' \tan\theta}$ respectively (Figure 8.28). When the length-scale is introduced into each parameter and Ri_{Ls}^{-1} and F are plotted (Figure 8.29) each point moves in or out along a ray passing from the origin through that point. Ri_{Ls}^{-1} is used for plotting purposes since mixing increases with Ri_{Ls}^{-1}.

As can be seen from a comparison of Figures 8.28 and 8.29 the spurious correlation has not significantly affected the interpretation of the results. There is a significant degree of scatter in the experimental results which can not be explained simply on the basis of experimental error. The relationship between F and Ri_{Ls}^{-1} appears to be slightly nonlinear; however this interpretation is somewhat dependent on the inclusion or exclusion of specific data points. In fact the data shown in Figure 8.28 could be interpreted as having a group of twenty-three points for which there is a linear relationship between $\frac{Q_f}{U_1 B_i}$ and $\frac{U_1^2}{g' \tan\theta}$ and a further three groups of three points each which fall below this line. However there are no obvious differences in any of the experimental variables for these three groups nor any indications of a difference from the qualitative observations made during the experiments. The data must therefore be treated as one group and it will be assumed that the relationship between F and Ri_{Ls}^{-1} is linear.

8.7.3 COMPARISON OF LABORATORY AND FIELD RESULTS.

Detailed monitoring of the flushing of saline pools during flow events at Lower Norton 2 and Tarranyurk was performed as part of this project. The collection and preliminary analysis of data at Lower Norton 2 were discussed in §8.7.1. Collection and processing of the Tarranyurk data used the same methodology as for Lower Norton 2 with the exception of obtaining stream discharge.

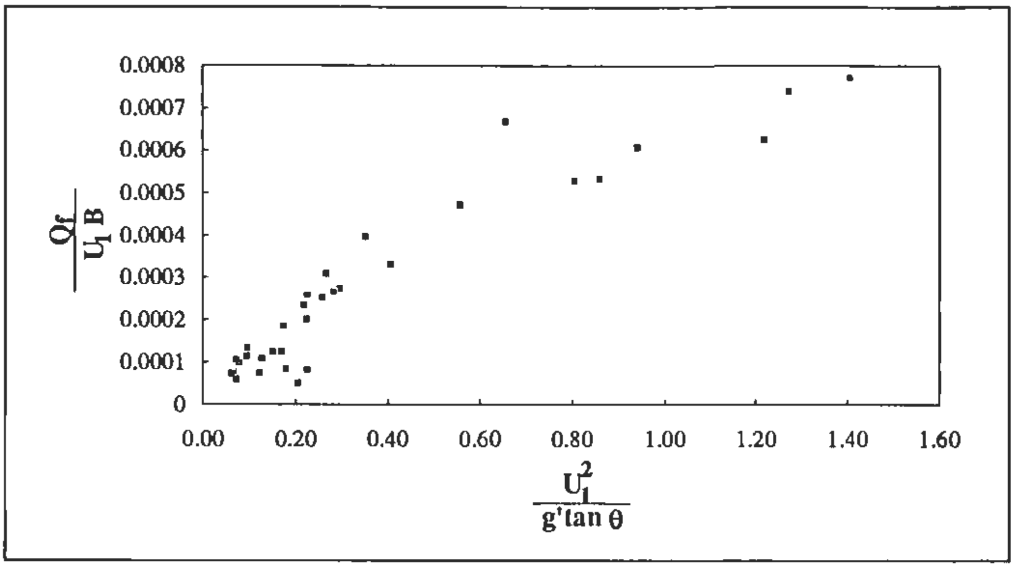


Figure 8.28: The relationship between $\frac{U_1^2}{g' \tan\theta}$ and $\frac{Q_f}{U_1 B}$ for laboratory experiments.

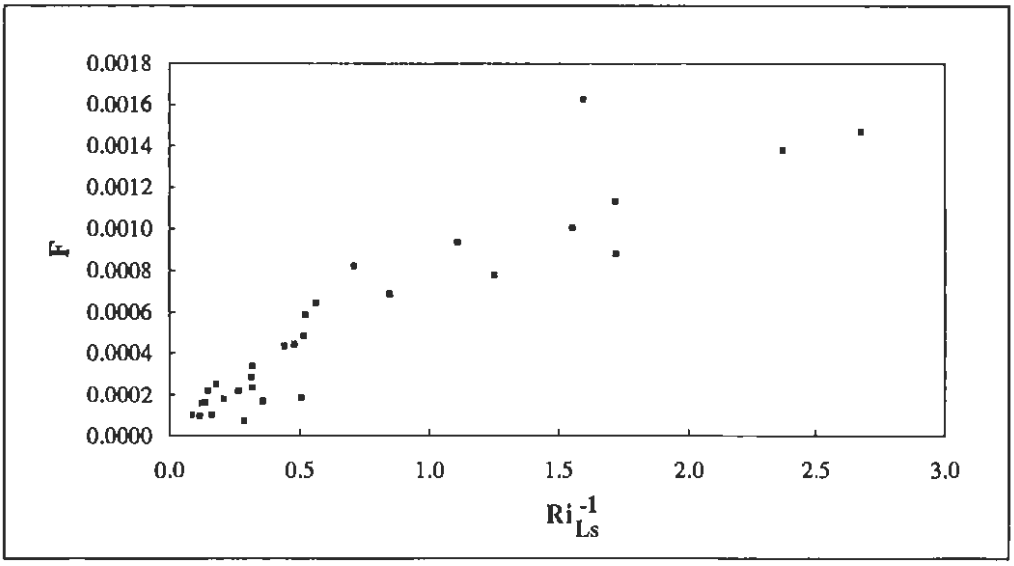


Figure 8.29: The relationship between F and Ri_{Ls} for the laboratory experiments.

8.7.3.1 Discharge at Tarranyurk.

River stage is monitored at station 415247 which is approximately 2 km upstream of the Tarranyurk saline pool site. However as no rating table exists for this station it is not possible to obtain flows directly from the measured stage. Both stage and discharge are measured at Lochiel (415246) which is located approximately 35 km upstream of 415247. No significant tributaries enter the

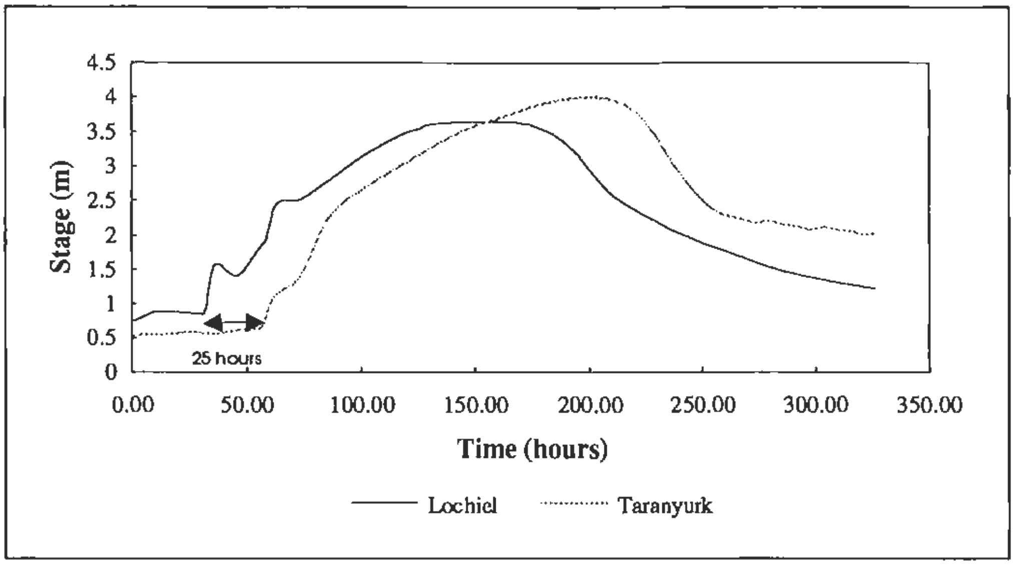


Figure 8.30: Comparison of stage hydrographs at Lochiel (415246) and Tarranyurk (415247). Time is hours since 0000 hours on the 1/9/93.

river between these two gauging stations. To obtain flows from the stage at 415247 an approximate rating curve for the event was developed as follows.

The stage hydrographs at 415246 and 415247 were compared and distinctive features on the two hydrographs identified (Figure 8.30). Mixing at Tarranyurk was complete in the early stages of the event so this part of the event was emphasised when estimating the rating curve. The travel time between 415246 and 415247 was estimated at 25 hours on the basis of the beginning of the event which was clearly identifiable. Based on this travel time and the distinctive features of the hydrograph five matching points were identified. These included the hydrograph peak, two low flow points before the beginning of the event and two points on the rising limb. The assumption that each pair of points represented the same discharge was made and the discharges were then simply transferred from 415246 to 415247. A curve was fitted to the stage discharge pairs for 415247 (Figure 8.31) and used with stage measured at 415247 to estimate the flow at the saline pool site.

8.7.3.2 Field Observations.

To calculate Ri_{Ls} an estimate of L was required. Estimates of L at different elevations were obtained from contour maps for each site. Values of L were then interpolated from these values using the elevation of the interface determined from the measured profiles. B_i was calculated as A_p/L where A_p is the plan area of the interface. The value of the downstream slope of the scour depression at each of these sites was also determined from the contour maps. The velocity, U_{1D}, of the

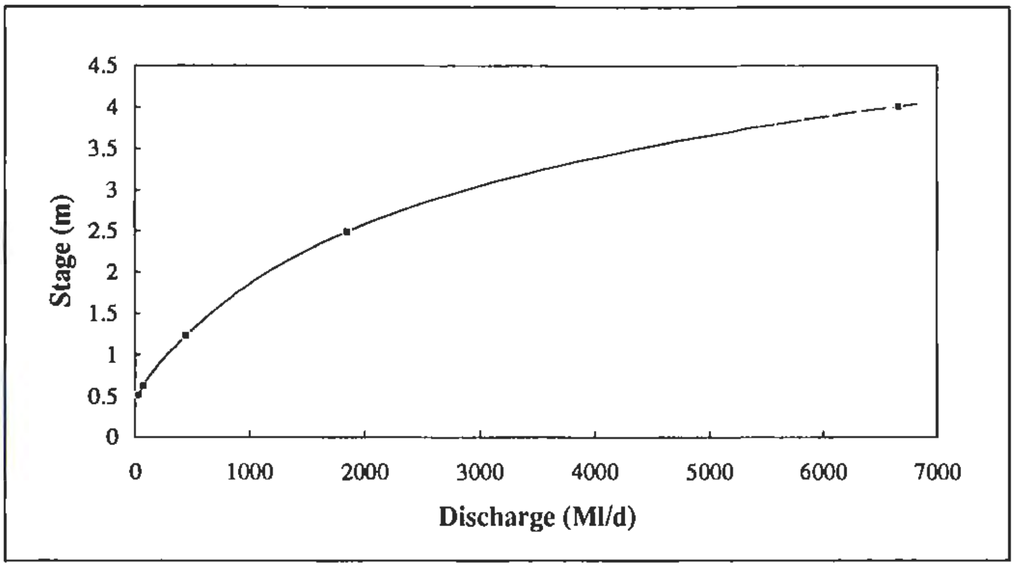


Figure 8.31: Estimated stage discharge relationship for the Wimmera River at Tarranyruk 415247.

overflowing layer was determined from the cross-sectional area at the point where the downstream outflow began. The velocity at this location is significant because this is the point where the shear stress due to the overflow must start to lift the saline fluid from the depression.

Figure 8.32 shows the relationship between the Richardson Number and the flushing parameter at Lower Norton 2 and Tarranyurk. During both these events the velocity of the overflow and the density difference remained approximately constant while L decreased as the flushing proceeded. This decrease in L was the main reason for the decrease in Ri_{Ls} (and increase in Ri^{-1}_{Ls}) It is noted that the relationship between F and Ri^{-1}_{Ls} is slightly non-linear. A possible explanation for this is a change in the relative importance of saline fluid removal by turbulent entrainment and the downstream outflow.

Assume that the flushing due to the non-dimensional downstream outflow, $F_D = \frac{Q_{fD}}{U_{1D}\, L\, B_i}$, in which Q_{fD} is the discharge of the downstream outflow is proportional to Ri^{-1}_{Ls}. The downstream outflow per unit width then becomes:

$$\frac{Q_{fD}}{B_i} = \frac{U^3_{1D}}{g' \tan\theta} \ . \tag{8.4}$$

On the one hand, if the velocity, density difference and downstream slope all remain approximately constant, the downstream outflow decreases with the width.

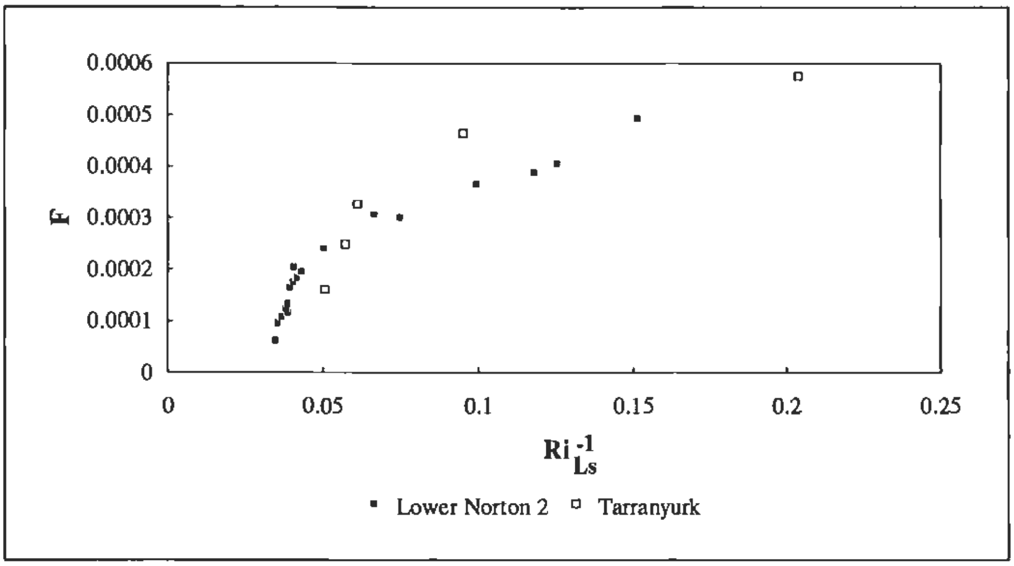


Figure 8.32: Comparison of relationship between F and Ri_{Ls} for mixing events at Lower Norton 2 and Tarranyurk.

On the other hand, assuming a constant entrainment velocity, the total rate of saline water removal by turbulent entrainment would decrease as the interfacial area decreases. This decrease in afflux due to turbulent entrainment may be even greater than expected simply on the basis of the reduction in interfacial area since turbulent entrainment is observed to increase along the interface; at least up to the point where the interfacial wave field is fully developed. It would therefore be expected that turbulent entrainment would become relatively less important and the slight non-linearity observed would result. This effect remains small because turbulent entrainment is only responsible for a small proportion of the total fluid removal.

8.7.3.3 Comparison with Laboratory Observations.

Figure 8.33 shows a comparison of the observations of flushing at Lower Norton 2, Tarranyurk and the laboratory experiments. It is obvious that the flushing rates observed in the field are significantly greater than those expected on the basis of the laboratory results. A number of factors may contribute to this difference.

There is a slight difference in both the velocity and the length-scales used in the analysis of the field data compared with those used in the analysis of the laboratory data. In the case of L, the maximum length of the interface was used as the length for the field analysis. Because of the shape of a typical scour depression this length will be greater than the mean length. However, in the laboratory experiments, the interface was rectangular. Therefore the value of L used was the mean interface length. An estimate of the ratio of the maximum to the mean

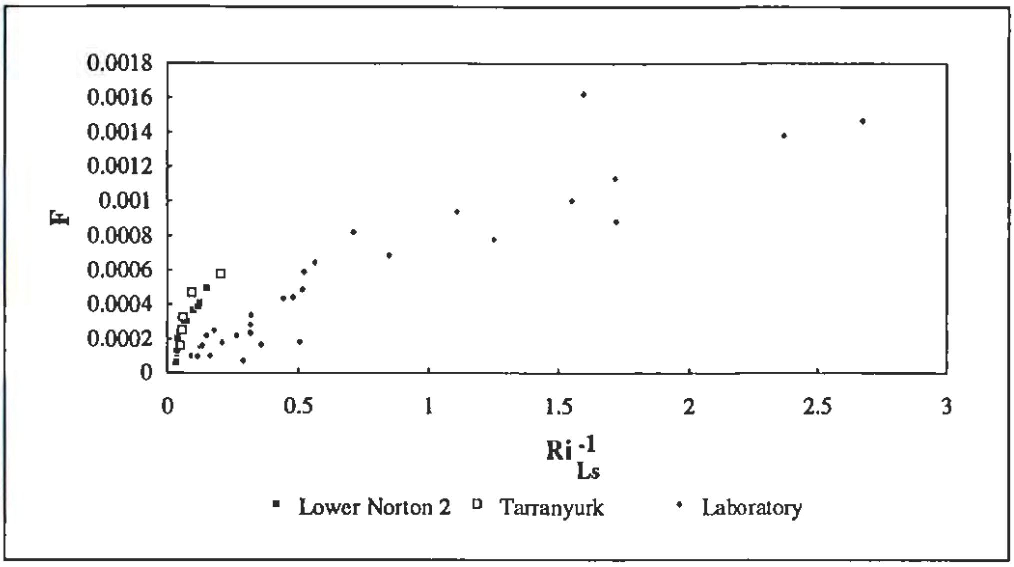


Figure 8.33: Comparision of relationship between F and Ri_{Ls} for laboratory and field mixing data.

length, R_l of a scour depression can be obtained by approximating the shape of the scour depression with an ellipse. In that case $R_l = 4/\Pi$.

The velocity-scale used in field and laboratory analysis was the cross-sectional mean velocity. In the laboratory experiments the flume cross-section was rectangular. This means that the horizontal velocity profile in the flume would be almost uniform except within the boundary layers at either wall. However, the depth of flow in the river changes across the cross-section. This leads to a non-uniform horizontal velocity profile in which higher velocities are found in the deeper parts of the channel. Of course the deeper parts of the channel are also those that are stratified. The average velocity within the stratified section of the channel would therefore be a more appropriate scale.

An estimate of the average velocity, U', within the stratified section of channel may be obtained as follows. It is assumed that the flow friction can be described using Manning's equation and n is constant across the cross-section. The cross-section is divided into small subsections and the proportional contribution of each subsection to the total conveyance of the cross-section is calculated using:

$$K_i' = \frac{A_i R_i^{2/3}}{\sum_{i=I}^{n} A_i R_i^{2/3}} . \qquad (8.5)$$

In Equation 8.5 A_i is the area of the subsection and R_i is the hydraulic radius of the subsection calculated using the length of solid boundary as the wetted perimeter.

It was assumed that the flow was unstratified while making these calculations. U' is then calculated as:

$$U' = \frac{Q\_K_i'}{A'} , \tag{8.6}$$

where the sum is taken over the stratified part of the cross-section and A' is the area of the stratified part of the channel above the halocline. These calculations resulted in a velocity-scale for the observed data that was between 11% and 18% higher than the cross-sectional mean velocity at Lower Norton 2 and 20% higher at Tarranyurk. Since the downstream outflow rate depends on U^3/L, these differences in length and velocity-scale between the laboratory and field could account for a field flushing rate twice those expected from the laboratory results. However the difference between the laboratory and the field data indicates an increase in flushing by a factor of five. A different explanation must therefore be sought.

8.7.4 THE POTENTIAL EFFECT OF BENDS ON FLUSHING.

8.7.4.1 Flow Around a Bend.

Channel bends are known to induce secondary currents and to have significant effects on the lateral and vertical velocity distributions (Rozovskii, 1957). A significant number of studies of flow in bends have been reported in the literature and a comprehensive review is not attempted here. Studies dealing with bend flows include work on developing predictive models for these flows (eg. de Vriend, 1981; Jin and Steffler, 1993; Zhou et al., 1993; Johannesson and Parker, 1989; Yeh and Kennedy, 1993a) or the flow and associated sediment transport (eg. Falcon Ascanio and Kennedy, 1983; Odgaard, 1986; Shimizu et al., 1990; Yeh and Kennedy, 1993b). Laboratory (eg. Rozovskii, 1957; Struiksma et al., 1985; Odgaard and Berg, 1988) and field (eg Dietrich and Smith, 1983) studies have also made significant contributions to the understanding of flow in bends.

Yeh and Kennedy (1993a) provide a detailed discussion of the dynamics of flows through bends on which the following description is based. The case of a fully developed bend flow is discussed first. Within a bend a transverse surface slope develops due to the radial acceleration. This surface slope results in a pressure force towards the centre of curvature which, assuming a hydrostatic pressure distribution, is uniform along any vertical. The stream-wise velocity, v, is however vertically non-uniform; being small near the bed and large near the surface. This means that the radial acceleration, which is given by v^2/r where r is the radius of

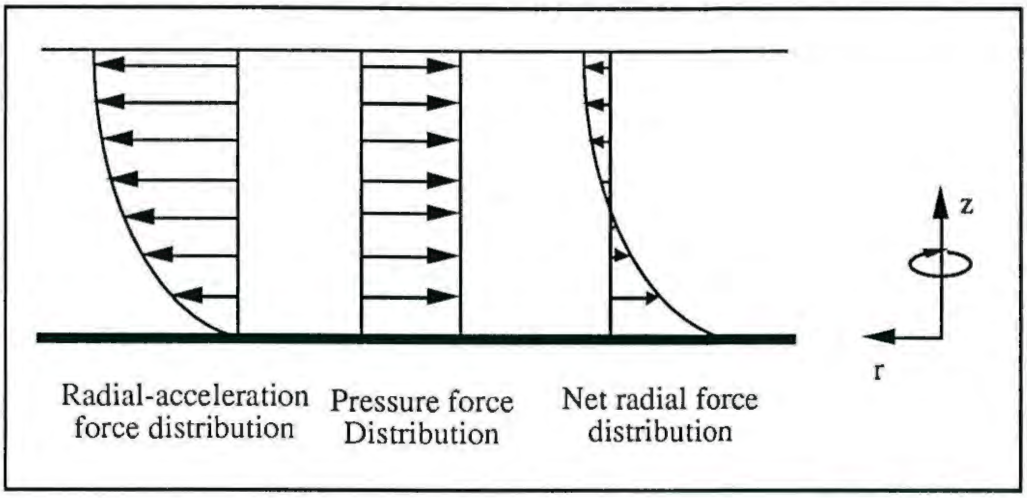


Figure 8.34: Vertical distrbution of pressure and radial inertia forces in flow around a bend.

curvature, is non-uniform and hence the force required to produce the radial acceleration is non-uniform. These force distributions and their resultant are illustrated in Figure 8.34.

As a result of the local force imbalance a moment, referred to as the centrifugal moment, exists which results in a secondary circulation such that there is a velocity away from the centre of curvature at the surface and towards the centre of curvature at the bed. In any channel of finite width, the presence of banks or walls then induces an upwelling in the inner part of the bend and a downwelling in the outer part. A helical flow results (Rozovskii, 1957; Falcon Ascanio and Kennedy, 1983; Yeh and Kennedy, 1993a). A component of bed shear stress towards the centre of curvature exists due to the secondary circulation. This bed shear stress produces a moment opposing the secondary circulation (Yeh and Kennedy, 1993a). Given the role of bed shear in retarding the secondary circulation and the role of radial acceleration in producing the flow, an important characteristic of bend geometry is the ratio of the depth to the radius of curvature. The secondary flow also alters the lateral stream-wise velocity profile by advecting stream-wise momentum toward the outer bank (Rozovskii, 1957; Yeh and Kennedy, 1993a).

Interactions between the helical flow and primary flow affect both of these flows. These interactions are best demonstrated by considering the angular (moment of) momentum about the centroid of a control volume consisting of a radial slice of the channel (Figure 8.35). The primary flow has a nonuniform velocity profile and therefore an angular momentum $\mathbf{M_r}$ about the radial axis at the entrance to the control volume . The secondary flow has an angular momentum $\mathbf{M_\theta}$ about the stream-wise axis at the entrance to the control volume.

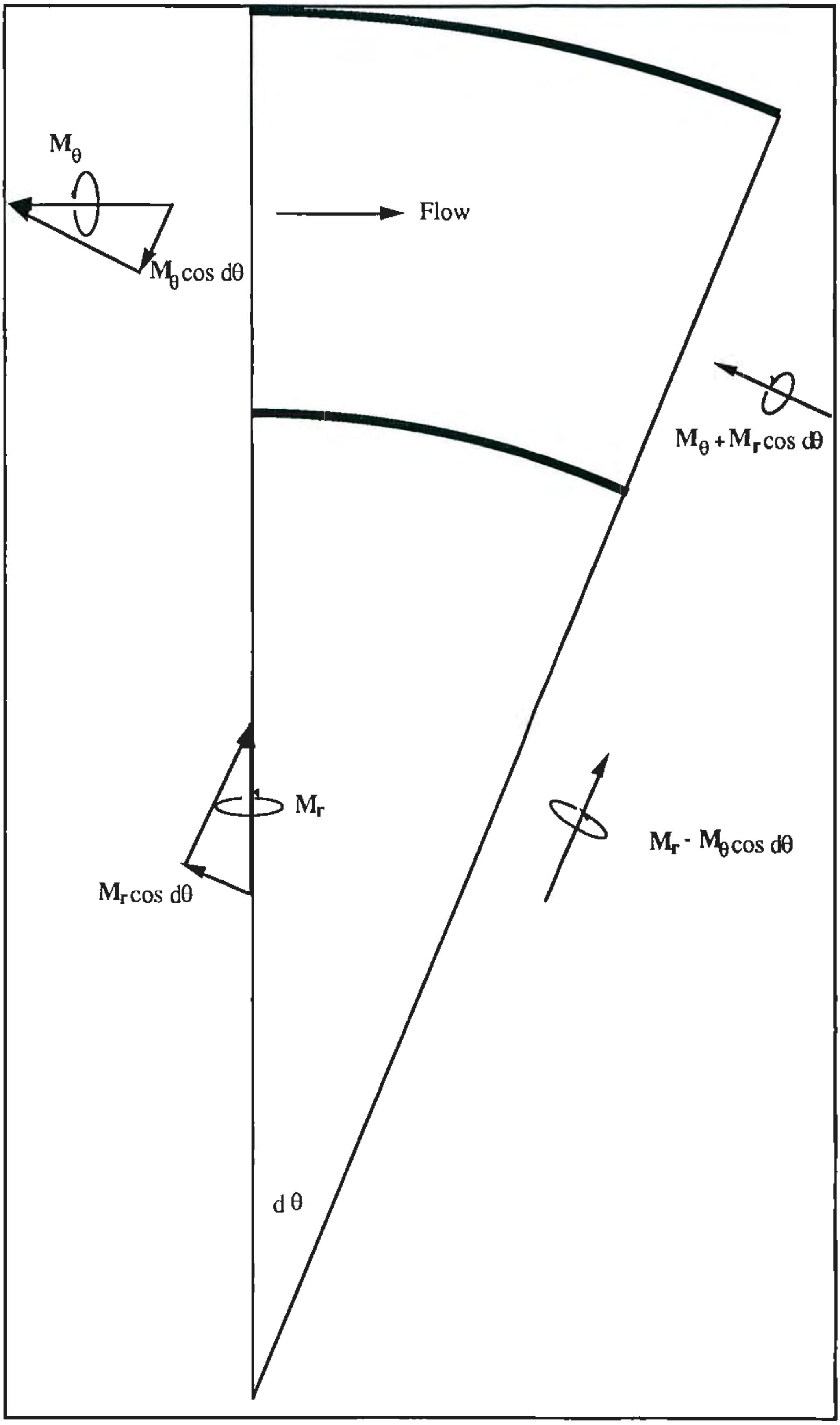


Figure 8.35: Fluxes of radial and streamwise angular momentum in flow around a bend.

Relative to the stream-wise axis at the exit of the control volume, $\mathbf{M_r}$ has a nonzero component equal to $\mathbf{M_r}$ cos $d\theta$ (Figure8. 35). This component is transferred from the primary to the secondary flow over the angular distance $d\theta$ in such a way that the angular momentum of the secondary flow is augmented. In other words part of the angular momentum resulting from the nonuniform vertical distribution of primary velocity transfers to the helical flow as a result of the change in flow direction (Yeh and Kennedy, 1993a).

A transfer of angular momentum from the helical flow to the primary flow also occurs due to the change in direction; however in this case the transfer actually reduces the angular momentum of the primary flow. That is the primary flow becomes more uniform due to a reduction of stream-wise velocity near the surface and an increase in stream-wise velocity near the bed. The action of shear stresses and a small pressure difference between the inner and outer walls induce moments that increase the angular momentum of the primary flow and decrease that of the secondary flow (Yeh and Kennedy, 1993a).

Fully developed channel flow around a bend can therefore be characterised as follows. A transverse surface slope is induced by the radial acceleration, which when coupled with a nonuniform vertical velocity distribution and the presence of banks, induces a secondary circulation with an outward velocity near the surface and an inward velocity near the bed. A component of bed shear stress towards the centre of curvature acts retard the secondary circulation. As a result of a kinematic effect arising from the change in direction; the secondary circulation is augmented and the vertical velocity profile becomes more uniform.

Turning now to a consideration of the development and decay of bend flow in an open channel (Figure 8.36). The transverse surface slope quickly becomes established as the flow enters the bend (Rozovskii, 1957). This leads to an increase in the stream-wise water surface slope near the inside of the bend and a reduction near the outside. As a result the flow near the inside of the bend accelerates and the velocity near the outside of the bend decreases. Thus the flow becomes concentrated near the inner bank and a translational secondary flow towards the inside of the bend develops to maintain continuity.

The transverse surface slope leads to the development of the centrifugal moment. This moment and the kinematic effect drive the secondary circulation which becomes stronger through the upstream part of the bend. This secondary circulation advects stream-wise momentum toward the concave bank. On the other hand the transverse secondary flow which developed at the start of the bend

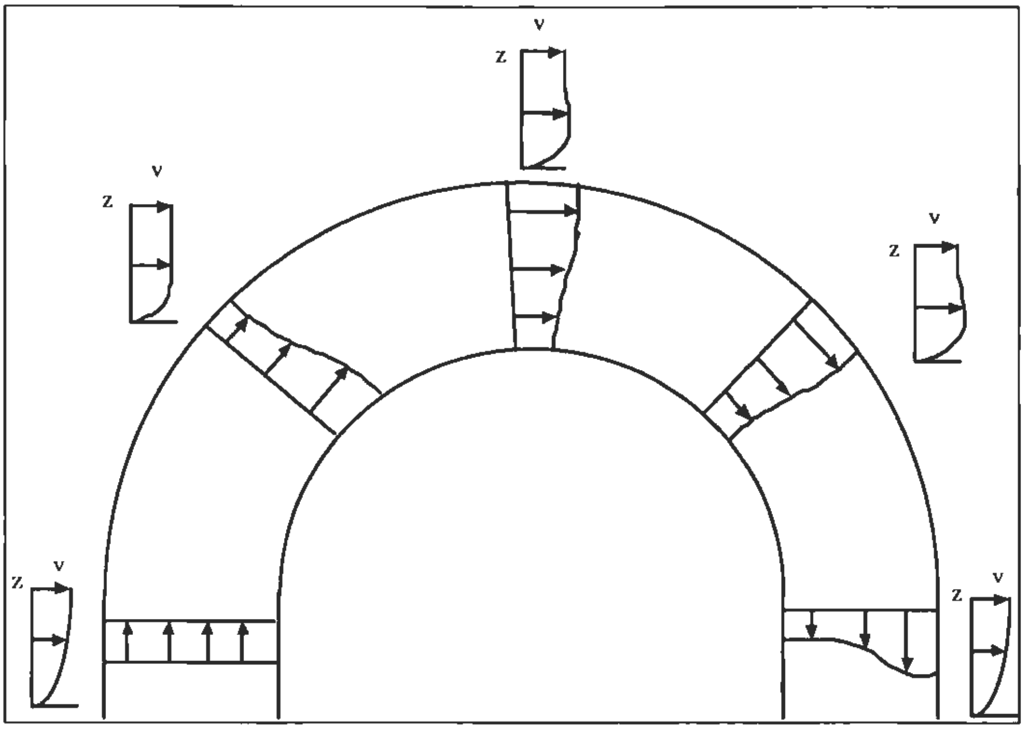


Figure 8.36: Changes in lateral and vertical velocity profiles around a bend.

advects momentum toward the convex bank. The secondary circulation is usually the stronger of these two in which case the lateral velocity profile gradually changes so that the flow moves toward the outer bank and an outward transverse secondary flow develops to maintain continuity. In very sharp bends the flow can remain concentrated near the inner bank (Yeh and Kennedy, 1993a).

As the secondary circulation develops, the kinematic effect produces a flatter vertical velocity profile which in turn leads to a reduction in the centrifugal moment. At the same time the secondary flow is augmented by angular momentum transferred from the primary flow. The retarding moment acting on the secondary circulation due to bed shear increases as the circulation strengthens. The reduction in the centrifugal moment coupled with the inertia of the secondary circulation and the transfer of angular momentum to the secondary circulation can cause the secondary circulation to overshoot its equilibrium strength and then diminish toward its equilibrium strength (Yeh and Kennedy, 1993a).

Once it has reached its equilibrium strength the secondary flow continues until the end of the bend and the lateral and vertical velocity profiles remain constant. The transverse secondary flow also disappears. At the end of the bend the transverse surface slope disappears leading to an increase in velocity at the outer bank and a decrease at the inner bank. An outward transverse secondary flow develops to maintain continuity at this point. The secondary circulation decays approximately exponentially downstream of the bend (Yeh and Kennedy, 1993a).

The above discussion indicates that the effects of inertia on secondary circulation can be significant. This leads to a lag between the change in curvature and the development of the secondary flow (Struiksma et al., 1985; Odgaard, 1986) which Odgaard (1986) suggested could be as much as 1/4 of a meander length (defined as the along channel meander wave length) in natural streams. Large phase lags have been observed in laboratory flows (eg. Odgaard and Bergs, 1988); however analytical modelling by Johannesson and Parker (1989) indicates that this phase lag may be small in rivers due to higher friction factors and/or longer meander length to water depth ratios in comparison to laboratory studies. Work by Zhou et al. (1993) supports this conclusion. At the moment field data on this phenomenon is limited and further field studies are required to clarify this issue.

8.7.4.2 Implications for Mixing.

The above discussion describes the characteristics of bend flows and their development and decay. From the perspective of flushing of saline pools the two features most likely to be important are the change in direction of the currents near the bed and the increase in velocity near the bed. It has been argued above that the length-scale of importance in determining the downstream outflow from a scour depression is the stream-wise length of the interface. Given that scour depressions in the Wimmera River are long and narrow; a change in flow direction near the bed so that the flow moved obliquely across the interface could significantly reduce this length-scale and lead to an increase in mixing (Figure 8.37).

Another and perhaps more important reason for expecting enhanced mixing on bends is the change induced in the vertical velocity profile. The velocity immediately above the interface is presumably more important than the depth averaged velocity in determining the shear stresses transferring momentum to the saline layer. Therefore the increases in near bed velocity associated with bend flows would be significant; especially since the downstream outflow from the saline pool increases with the cube of the velocity. While a quantitative analysis of these effects is beyond the scope of this thesis; it is considered that the change in direction of near-bed currents and especially the increase in near-bed velocities have the potential to explain the enhanced flushing observed in the river compared to the laboratory. This enhancement would be expected to increase with the ratio of the depth to the radius of curvature.

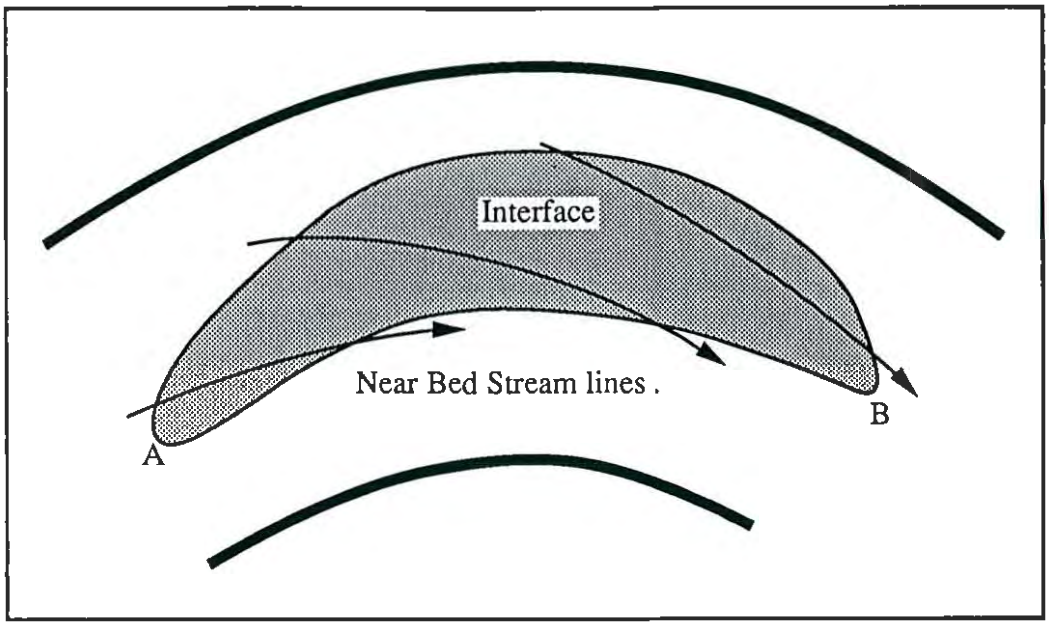
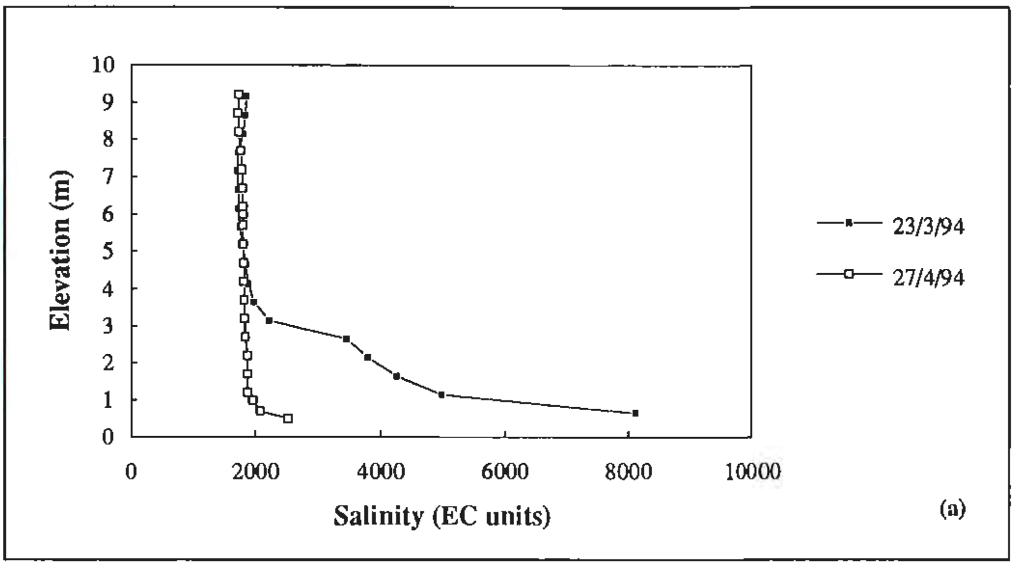


Figure 8.37: Reduction of interface length due to secondary circulation in a bend.

8.8 MIXING OF SALINE POOLS BY OTHER MECHANISMS.

Figure 8.38 shows vertical temperature and salinity profiles measured at Big Bend on the 23/3/94 and the 27/4/94. The stream discharge at Big Bend was low during this period (Figure 8.39) and had been less than 100 Ml/d since the 24/11/93. Figure 8.40 shows continuously monitored temperature and salinity 0.5 m and 5.0 m from the stream bed for the period 23/3/94 to 27/4/94. The vertical grid lines represent midnight. On the 4/4/94 the temperature at 5 m rises slightly then a period of cooling begins on the 7/4/94 and continues until the 13/4/94. Significant cooling near the bed began on the 13/4/94 and continued until the 15/4/94. Further cooling at 0.5 m occurred on the 19/4/94 and on the 22/4/94 and the 23/4/94 when the water column became fully mixed. The salinity data (Figure 8.40) show a slight reduction in salinity occurring between the 11/4/94 and the 13/4/94 then rapid reductions associated with the periods of cooling.

Figure 8.41 shows the minimum and maximum daily air temperatures for this period at Longerenong which is 40 km away from the site but still on the Wimmera Plains. There is a general cooling of approximately 5° C in both the daily minimum and daily maximum temperatures over the period when mixing was observed together with significant fluctuations from day to day. The fact that mixing is associated with a significant cooling of the water column and a general reduction in surface air temperature suggests that convection due to surface cooling is responsible for the mixing. The small warming at 5 m at the start of



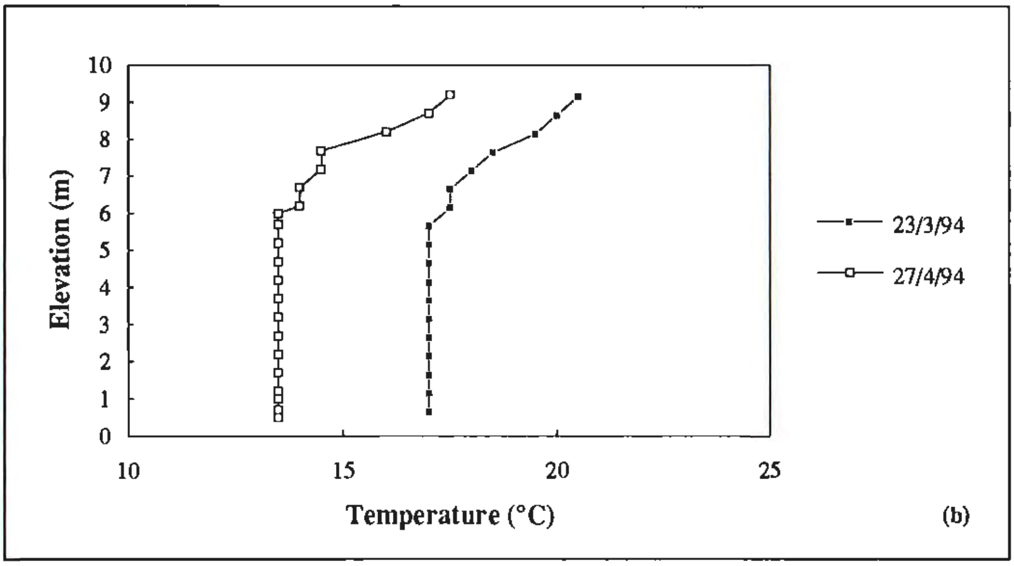


Figure 8.38: (a) Vertical salinity profiles at Big Bend on the 23/3/94 and 27/4/94. (b) Vertical temperature profiles at Big Bend on the 23/3/94 and 27/4/94.

mixing may have resulted from a small amount of heat being mixed down through the water column. On the 23/3/94 there was significant thermal stratification above 5 m which could have acted as the source of this heat (Figure 8.38).

Figure 8.42 shows temperature readings at Big Bend collected between 1990 and 1994. The annual cycle of temperature is characterised by maximum temperatures in summer and minimum temperatures in winter as expected. Cooling through the autumn period is a consistent phenomenon which indicates that mixing due to surface cooling may occur during the autumn of most years. In fact mixing due to surface cooling was also observed at Big Bend during the autumn of 1993. Strong

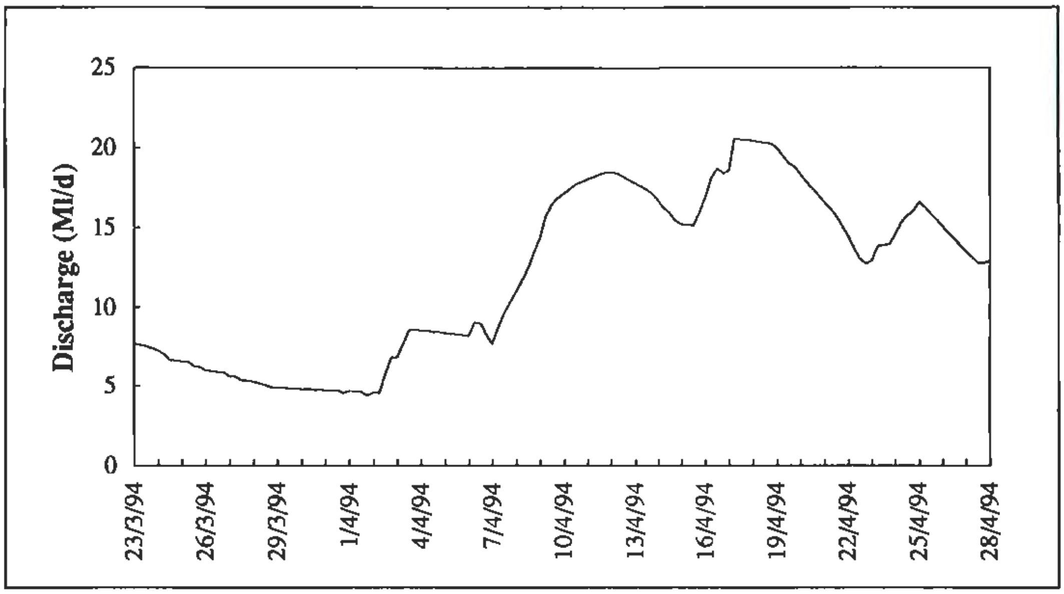


Figure 8.39: Discharge in the Wimmera River at Big Bend for the period 23/3/94 to 28/3/94.

salinity stratification, such as in the autumn of 1992 at Big Bend, may inhibit this mixing.

It is interesting to note that the stratification observed at Lower Norton 2 during the autumn of 1993 was significantly less than expected given the duration of low flow at that time (§ 9.2.2). Mixing due to surface cooling may also have been significant at this site during this period. Further investigation would be required to quantify the relative importance of this process in the Wimmera River.

8.9 FORMATION OF STRATIFICATION.

There are several important aspects of the formation of stratification in saline pools that are discussed below. These include the process by which the water column becomes stratified, the conditions under which stratification begins to develop and the rate at which stratification develops.

8.9.1 FORMATION MECHANISMS.

Anderson and Morison (Anderson, 1989c; Anderson, 1989g) suggest two mechanisms by which stratification can develop. Firstly, groundwater seeping directly into the stream channel may collect in depressions in the stream bed. Evidence supporting this hypothesis includes similarities between the ionic composition and temperature of groundwater and the saline water below the halocline and consistent lower layer salinities between stratification episodes. Anderson and Morison refer to this as primary stratification.

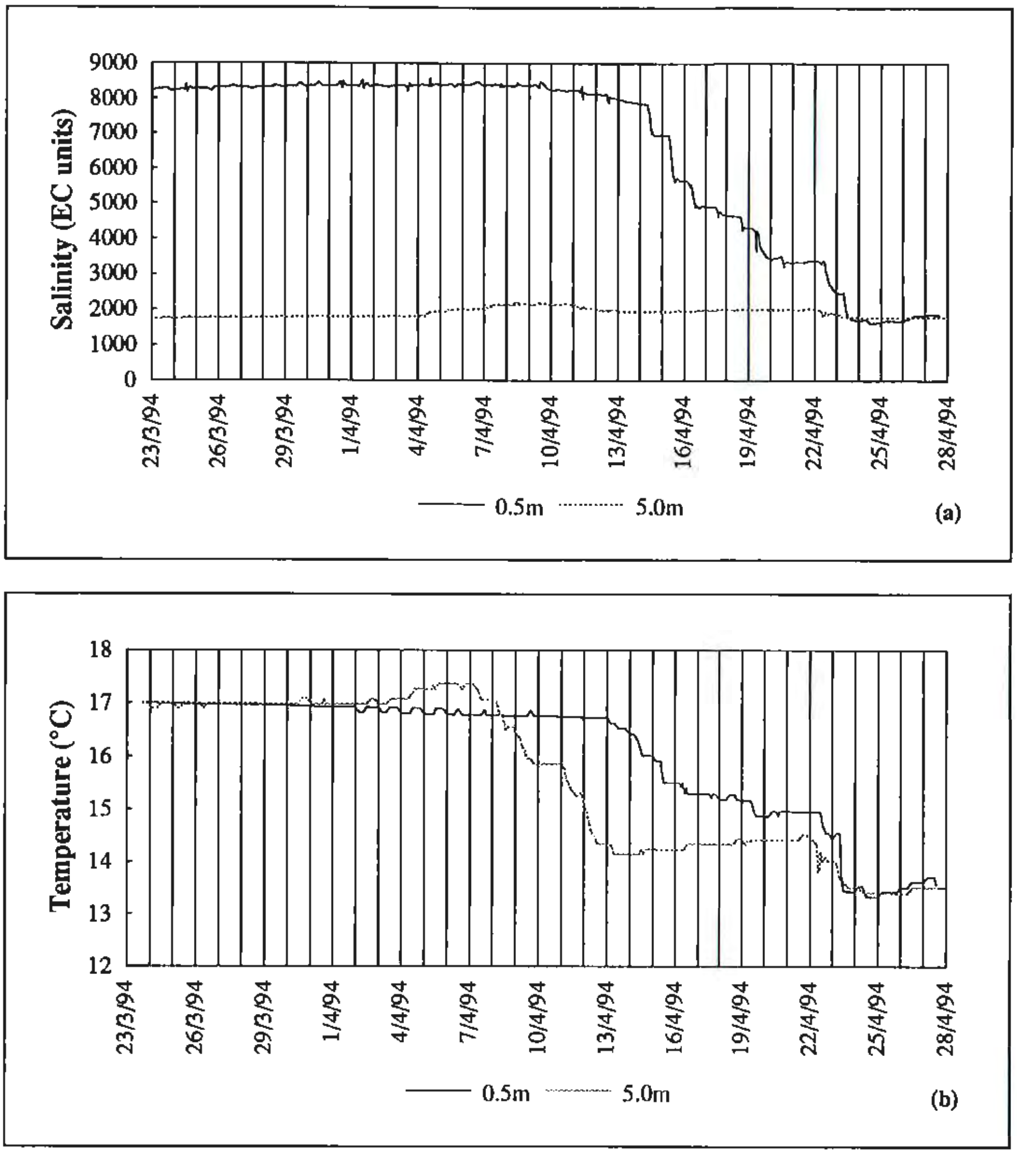


Figure 8.40: (a) Variation in salinity at elevations of 0.5 and 5.0 m in the Wimmera River at Big Bend for the period 23/3/94 to 28/4/94. (b) Variation in temperature at gauge heights of 0.5 and 5.0 m in the Wimmera River at Big Bend for the period 23/3/94 to 28/4/94. Vertical gridlines represent midnight.

Secondly, saline water flowing down the stream channel from upstream could potentially deposit in scour holes. This requires a source of saline water upstream of the saline pool sites. Anderson and Morison observed this process at Big Bend in July, 1988. Following an extensive low flow period the vertical salinity profile was almost uniform and the salinity was 12 000 EC. A small flow event at the end of this low flow period increased the bottom salinity to 25 000 EC and decreased the surface salinity to 5 000 EC. A halocline was present 6 m from the bed of the stream and the total water depth was 9.5 m. Significant volumes of saline water had been observed in the reaches upstream of Big Bend prior to this event and

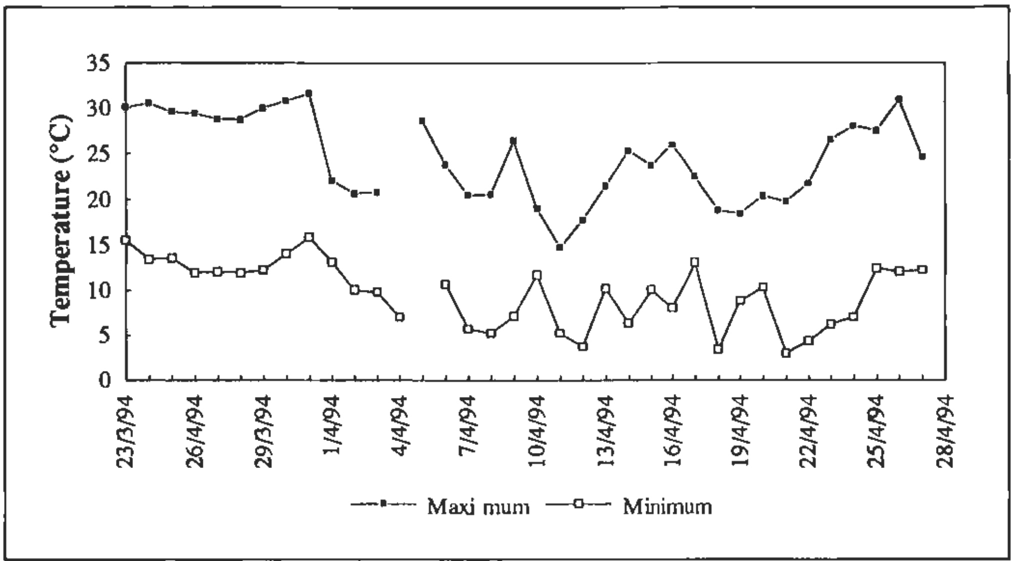


Figure 8.41: Variation in daily maximum and minimum air temperatures at Longerenong for the Period 23/3/94 to 28/4/94.

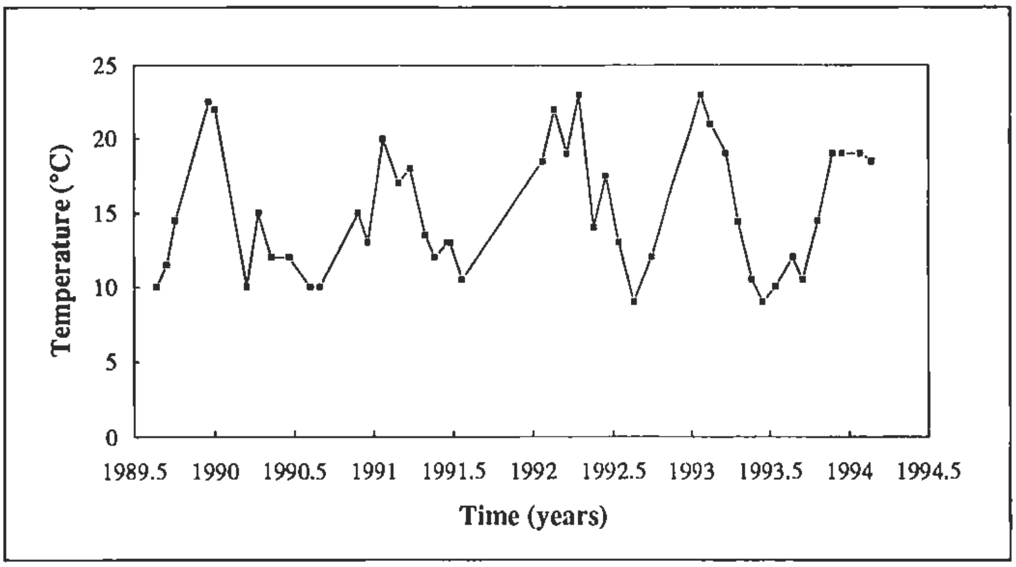


Figure 8.42: Seasonal variation of water temperature in the Wimmera River at Big Bend. Water temperatures were measured at an gauge height of 5 m.

Anderson and Morison (1989c; 1989g) hypothesised that this saline water had been transported down the stream as it began to flow and was partly deposited in the scour hole at Big Bend. This is referred to as secondary stratification. Anderson and Morison (1989c; 1989g) also suggested that distinct layers observed at other sites may also have resulted from similar mechanisms.

During this study a variation of the secondary stratification mechanism was observed at Lower Norton 1. Figure 8.43 shows salinity profiles on the 4/8/92 and the 13/8/92. The profile on the 13/8/92 was measured shortly after the

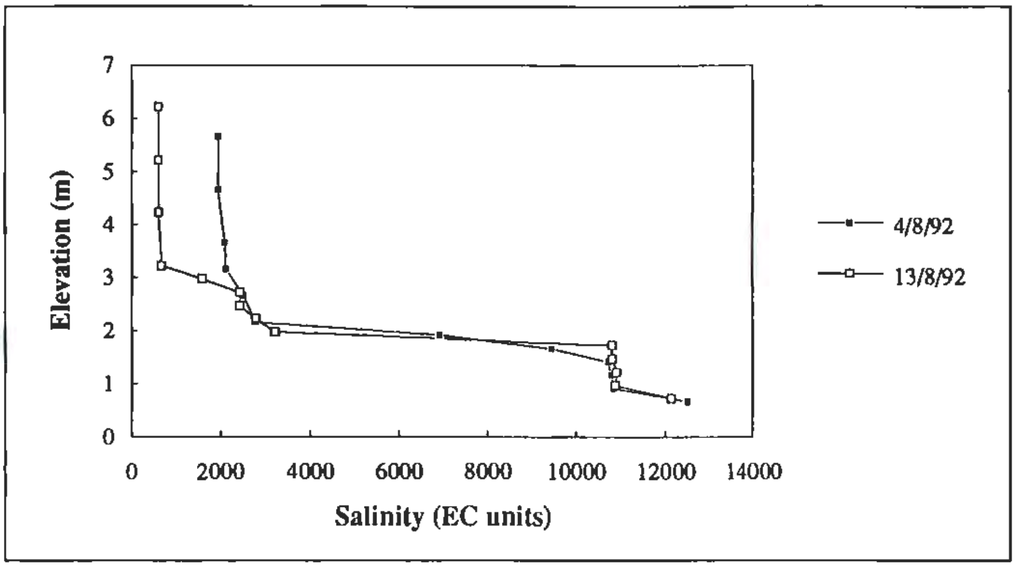


Figure 8.43: Vertical salinity profiles at Lower Norton 1 for the 4/8/92 and the 13/8/92.

commencement of a flow event when the discharge in the river was approximately 230 Ml/d and inflow was relatively fresh (600 EC). This fresh and less dense water had displaced the top 2.5 m of the original water and mixing was continuing Western et al. (1993). Subsequently the stream discharge increased and the water column at this site was mixed completely. This process has been suggested by Morrissy (1979) as a mechanism for the formation of saline pools observed in the Murray River in Western Australia. Thus buoyant inflows, whether they are more or less dense than the existing water in a river pool, can lead to the formation of stratification.

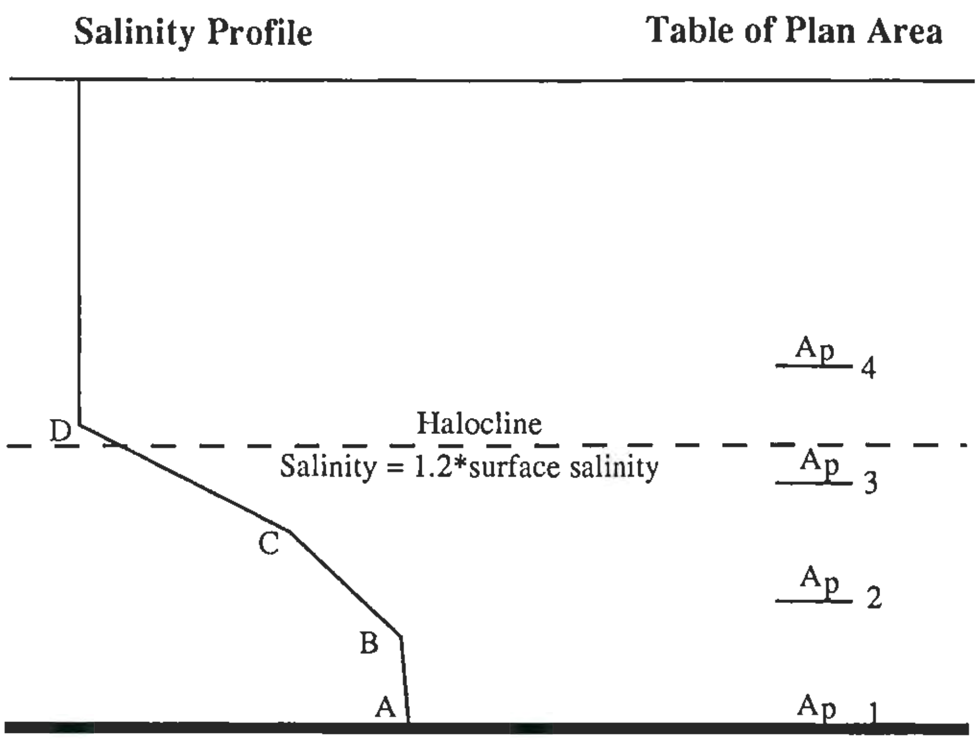
8.9.2 STRATIFICATION DUE TO GROUNDWATER INFLOWS.

8.9.2.1 Calculation of an Equivalent Groundwater Volume.

Before discussing the formation of stratification due to groundwater inflow it is necessary to consider how the degree of stratification can be quantified. Given that the interest is in the contribution of groundwater to the stratification; the amount of groundwater required to supply the additional salt stored below the top of the halocline is a relevant measure and was calculated as follows.

Figure 8.44 details the calculation of the volume, V_n, of the scour depression at points on a salinity profile. Equation 8.7 was used to calculate volume from plan area A_p.

$$V_n = \sum_{i=1}^{n-1} 0.5*(A_{p(i)} + A_{p(i+1)})*(h_{i+1} - h_i) \quad . \qquad (8.7)$$



Calculation of storage volume a tpoints on a measured salinity profile.

1. Volumes at points 1, 2,3 and 4 are calculated by integrating the plan area of the scour depression between the bed and each point using Equation 8.
2. Plan areas at A, B, C and the halocline are calculated by linear interpolation of values in the table of plan areas.
3. Storage volumes at A, B, C and the halocline are calculated by increasing the volume at the point in the table immediately below the point of interest on the salinity profile by an appropriate amaount. for example the volume below C is calculated as the volume at 2 plus $0.5*(h_C - h_2)*(A_{pC} + A_{p2})$ where A_{pC} and A_{p2} are the plan areas at C and 2 respectively and h_C and h_2 are the elevations at C and 2 respectively.

Figure 8.44: Calculation of equivalent groundwater volume.

The equivalent groundwater volume was then calculated using the volume and salinity at each point on the profile below the top of the halocline. The top of the halocline was defined as the point at which the salinity was 120% of the surface salinity. The water column was divided into slices bounded by each point on the profile (eg a slice between B and C) and the volume of groundwater was calculated using:

$$V_{gwj} = (V_1 - V_0) * \frac{0.5*(S_0 + S_1) - S_{surf}}{S_{gw} - S_{surf}} \quad . \qquad (8.8)$$

In Equation 8.8 V_{gwj} is the equivalent groundwater volume in slice j, V is the volume below the point on the profile, S is the salinity at the point on the profile and the subscripts 0 and 1 refer to bottom and top of the slice respectively. S_{gw} is the groundwater salinity and S_{surf} is the surface water salinity. The total equivalent volume of groundwater was then calculated by summing the contribution from each slice.

Equation 8.8 is obtained from a mass balance for water and salt within the slice assuming that the water within the slice is a mixture of groundwater with a salinity S_{gw} and surface water with a salinity S_{surf}. The surface water salinity was obtained from the top point on the profile and the groundwater salinity was estimated for each site from salinity profiles during strongly stratified periods. A problem exists with this assumption because the salinity of the surface water changes and thus the current surface salinity may not be representative of the surface water that was actually incorporated within the stratified section. However, since the variation in surface salinity is small compared with the difference between the surface and groundwater salinities, this is not considered to be a major problem. A similar criticism of and justification for the constant groundwater salinity assumption can be made.

Two additional assumptions were made to deal with specific problems in these calculations. Firstly, if the salinity profile did not extend to the bottom of the scour hole it was assumed that the salinity of the water below the lowest point on the profile could be represented by the salinity at the lowest point on the profile and Equation 8.8 was adjusted accordingly. Typically only small volumes were located below the lowest point on the profile. Secondly, in some cases, the halocline was actually above the top (ie. the elevation of the high point in the channel invert immediately upstream or downstream of the saline pool) of the scour depression. It is then possible for saline water to drain to other depressions. In that case the volume of water considered in the calculations was limited to that within the scour pool as it was not possible to define adequately a plan area. This only occurred at Lower Norton 1 and Polkemmet.

8.9.2.2 Initiation and Rate of Formation of Stratification.

Saline pools at Lower Norton 1 and 2, Polkemmet and Tarranyurk all form primarily as a result of groundwater flowing directly into the stream channel. Two data sets provide information on the initiation of stratification: continuous salinity

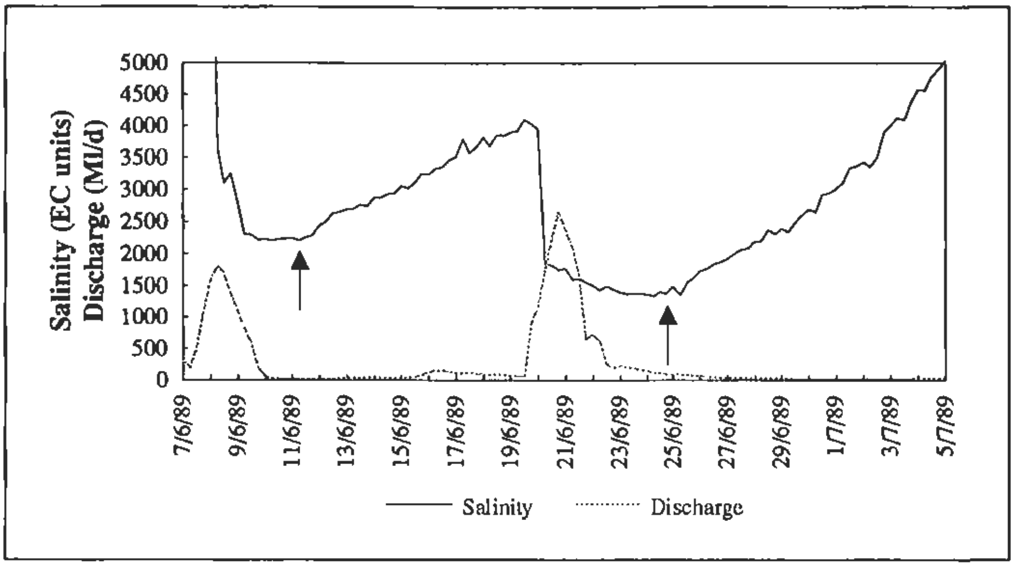


Figure 8.45: Continuously monitored salinity at an elevation of 0.8 m at Lower Norton 1 and discharge for the Wimmera River at Horsham for the period 7/6/89 to 5/7/89. Arrows indicate the detection of stratification following after the return of low river discharge.

monitoring at Lower Norton 1 and vertical salinity profiles at all four sites. Figure 8.45 shows typical continuously recorded salinity data from Lower Norton 1. The point in time at which the salinity starts to rise rapidly provides an estimate of the time when the halocline begins to pass the lowest salinity sensor. This point is noted on the graph and represents the time when there is approximately 50 m^3 of saline water below the halocline. Table 8.1 provides dates and times at which stratification was first detected from the continuous salinity record at Lower Norton 1. The discharge, Q, when stratification was detected, the peak discharge during the preceding flow event, Q_{peak}, the time elapsed since discharges of 250 Ml/d, T_{250} and 500 Ml/d, T_{500}, were observed and the length of time the flow exceeded 1000 Ml/d, T_{high} are also provided. Times are supplied to the nearest 6 hours. Discharges were obtained from the gauging station for the Wimmera River at Horsham (415200) and a travel time of 6 hours between Horsham and Lower Norton 1 was assumed. Discharges in the two tributaries, McKenzie and Norton Creeks, which enter the Wimmera River between Horsham and Lower Norton 1 were assumed to be negligible.

Figures 8.46, 8.47, 8.48 and 8.49 provide the first profiles measured after saline pools have begun to form at Lower Norton 1, Lower Norton 2, Tarranyurk and Polkemmet and the hydrographs for the four weeks preceding the profile measurement. Tables 8.2, 8.3, 8.4 and 8.5 provide discharges and hydrograph characteristics for each of the flow events preceding the measurement of the

Date	Time	Q (Ml/d)	Qpeak (Ml/d)	Thigh (hours)	T500 (hours)	T250 (hours)
11/6/89	0600	20	1810	36	42	36
24/6/89	1800	110	2650	42	54	48
1/10/89	1200	200	6360[a]	1440[a]	24	12
16/3/93	0000	80	610	0	114	84

Table 8.1: Times, discharges and hydrograph characteristics when stratification was detected by continuous monitoring at Lower Norton 1. [a] This was actually a small event at the end of an extended period of high flow. Statistics for the high flow period have been provided since it is felt that they are more representative of the flow conditions prior to the formation of stratification, The figures for the small event are 1350 Ml/d and 102 hours for Q_{peak} and T_{high} respectively.

Date	Time	Q (Ml/d)	Qpeak (Ml/d)	Thigh (hours)	T500 (hours)	T250 (hours)	Vgw (m^3)
29/10/92	1200	580	13020	1008	0	0	15
12/11/92	1430	200	13020	1008	126	6	15
12/1/93	1500	140	7520	222	222	102	22
11/2/93	0935	220	4060	144	84	0	3
22/3/93	1100	35	610	0	270	240	214
16/9/93	1130	16	5730	162	138	114	36
21/10/93		76	2560	54	288	246	146
24/11/93	0830	38	1620	66	294	234	2

Table 8.2: Times, discharges and hydrograph characteristics when stratification was detected by manual profiles at Lower Norton 1.

profile and the equivalent volume of groundwater stored below the halocline for each profile for the three sites. Flows at Lower Norton 2 were calculated in the same way as for Lower Norton 1. For Tarranyurk the discharge was obtained from Lochiel (415246) when available and a 24 hour travel time was assumed. When discharge was unavailable at Lochiel the station Upstream of Dimboola (415256) was used and a 36 hour travel time was assumed. Flows at Polkemmet were obtained from Wimmera River Upstream of Dimboola (415256) and a 12 hour travel time was assumed.

It is clear from Tables 8.1 to 8.5 that the stratification at all three sites re-establishes under continuous flows and that these flows can be as high as 200-300 Ml/d. On the 29/10/92 some stratification was present at Lower Norton 1 when the discharge was 580 Ml/d. As well as the stream discharge at which stratification re-establishes, the rate at which the stratification develops and the

Date	Time	Q (Ml/d)	Qpeak (Ml/d)	Thigh (hours)	T500 (hours)	T250 (hours)	Vgw (m^3)
12/1/93	1530	140	7520	222	222	102	7
3/3/93	1030	44	4060	144	564	480	109
21/4/93	1030	17	610	0	996	966	12
21/10/93	1610	72	2560	54	288	252	124
11/1/94	0800	28	1620	66	1416	1356	1180

Table 8.3: Times, discharges and hydrograph characteristics when stratification was detected by manual profiles at Lower Norton 2.

Date	Time	Q (Ml/d)	Qpeak (Ml/d)	Thigh (hours)	T500 (hours)	T250 (hours)	Vgw (m^3)
14/11/92	1454	300	600	0	114	-	1
9/12/92	1340	300	1050	30	144	-	1
5/3/93	1200	39	3880	132	540	498	240
19/3/93	1420	97	330	0	-	90	71
20/10/93	1700	88	2340	60	192	156	20
12/1/94	1200	28	1320	84	1386	1332	277

Table 8.4: Times, discharges and hydrograph characteristics when stratification was detected by manual profiles at Tarranyurk.

Date	Time	Q (Ml/d)	Qpeak (Ml/d)	Thigh (hours)	T500 (hours)	T250 (hours)	Vgw (m^3)
14/8/92	1008	77	5480	378	7602	7566	508
26/8/92	1930	95	2180	78	138	108	48
14/11/92	1235	182	12020	1014	156	30	17
7/12/92	1745	329	1120	48	150	0	5
3/3/93	1030	41	3880	132	528	486	210
20/10/93	0925	87	2340	60	228	192	37
8/12/93[a]	1655	13	1620	72	612	558	289

Table 8.5: Times, discharges and hydrograph characteristics when stratification was detected by manual profiles at Polkemmet. [a] Flow data obtained from Horsham.

consistency of the rate of development between events for a given site are also of interest. Factors that may cause a variation in the rate of formation of stratification include variations in the groundwater flow rate due to, for example, bank storage effects or seasonal fluctuations in water table elevations and mixing

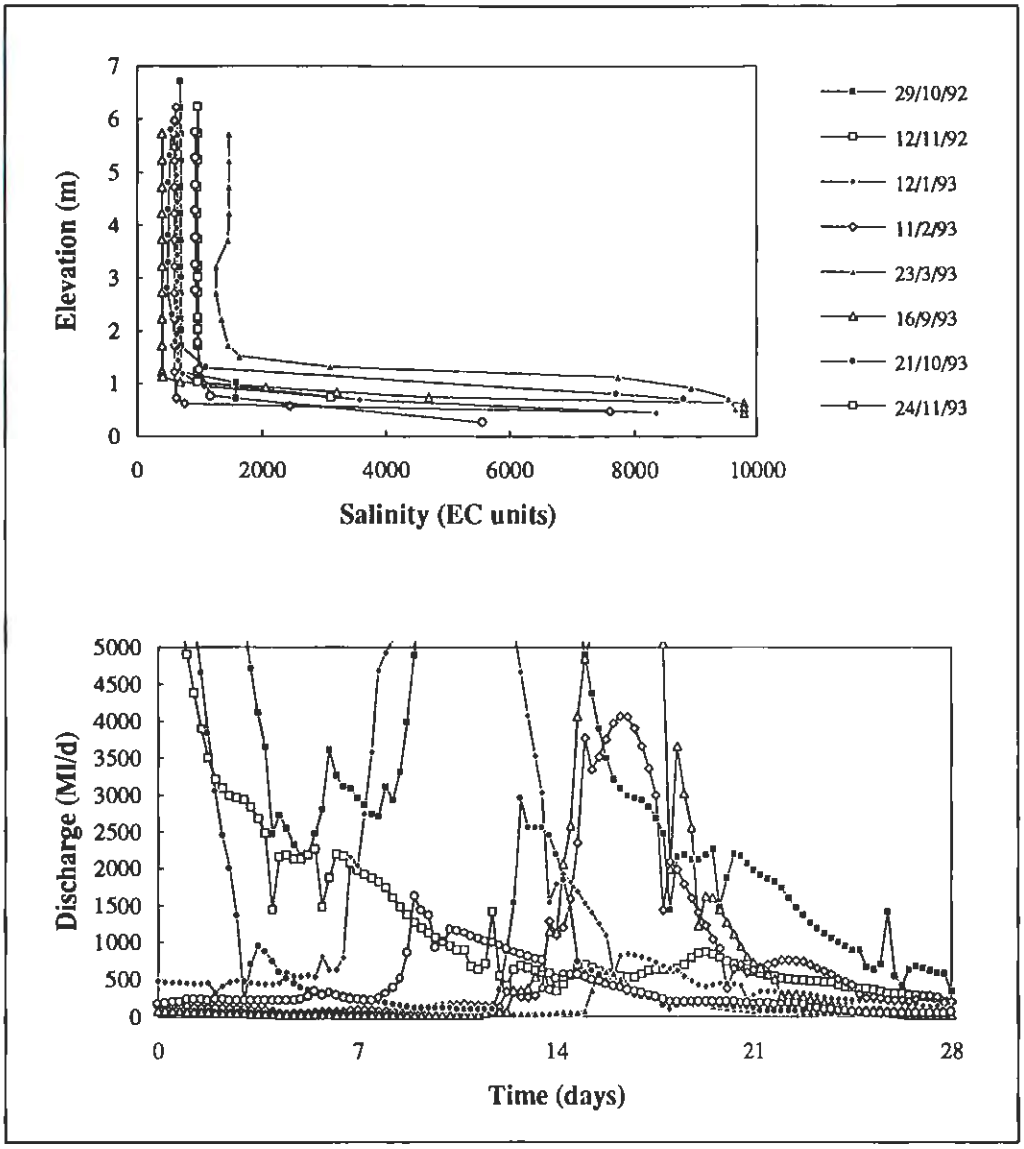


Figure 8.46: Vertical salinity profiles measured at Lower Norton 1 following the development of stratification and discharge in the Wimmera River at Horsham for the 28 days prior to the measurement of each profile.

due to low but significant stream discharges or other mixing mechanisms. The statistics provided in Tables 8.1 to 8.5 allow some comparison of rates of formation of stratification.

The peak discharge and period of time for which the flow exceeded 1 000 Ml/d are provided for the flow event preceding stratification. These are measures of the size and duration of the high flows. In Tables 8.1 to 8.5, T_{250} and T_{500} are indicative of the time elapsed since the river returned to low flow and hence the time over which the stratification has formed. Assuming the elevation of the continuous salinity sensor located near the bottom of Lower Norton 1 is constant,

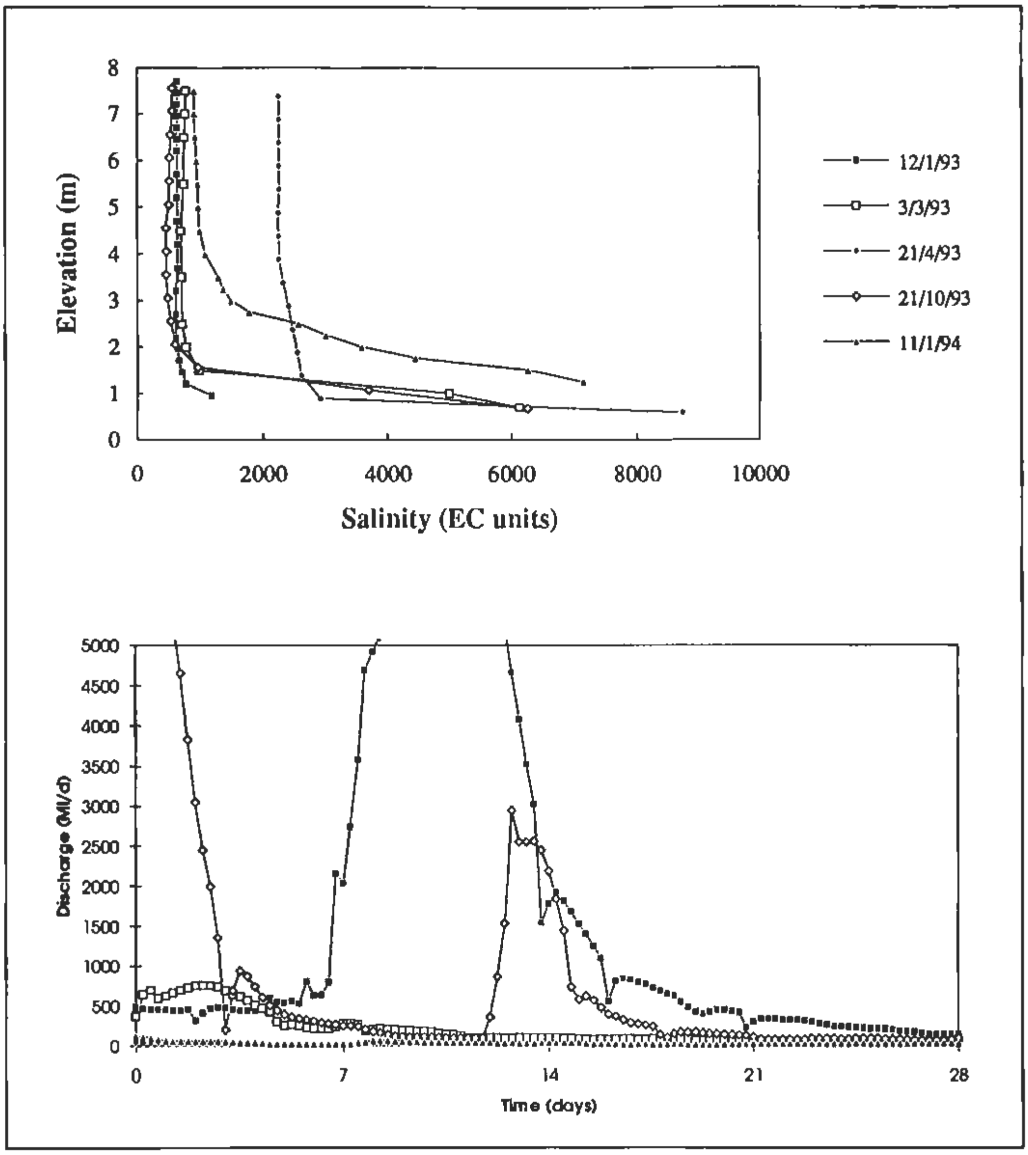


Figure 8.47: Vertical salinity profiles measured at Lower Norton 2 following the development of stratification and discharge in the Wimmera River at Horsham for the 28 days prior to the measurement of each profile.

T_{250} and T_{500} in Table 8.1 are indicative of the rate of formation of stratification. The shorter the time the more rapid the establishment of the stratification. In Tables 8.2 to 8.5 the rate of formation is indicated by the relationship between these times and the equivalent groundwater volume.

An examination of Table 8.1 indicates that stratification returns more rapidly after higher and more extensive periods of high flow. However the elevation of the salinity sensor is not constant. This is due to the anchor to which the sensor is attached sinking into the river bed and due to the removal of the salinity sensors and anchor for maintenance. The volume of water below the sensor varies

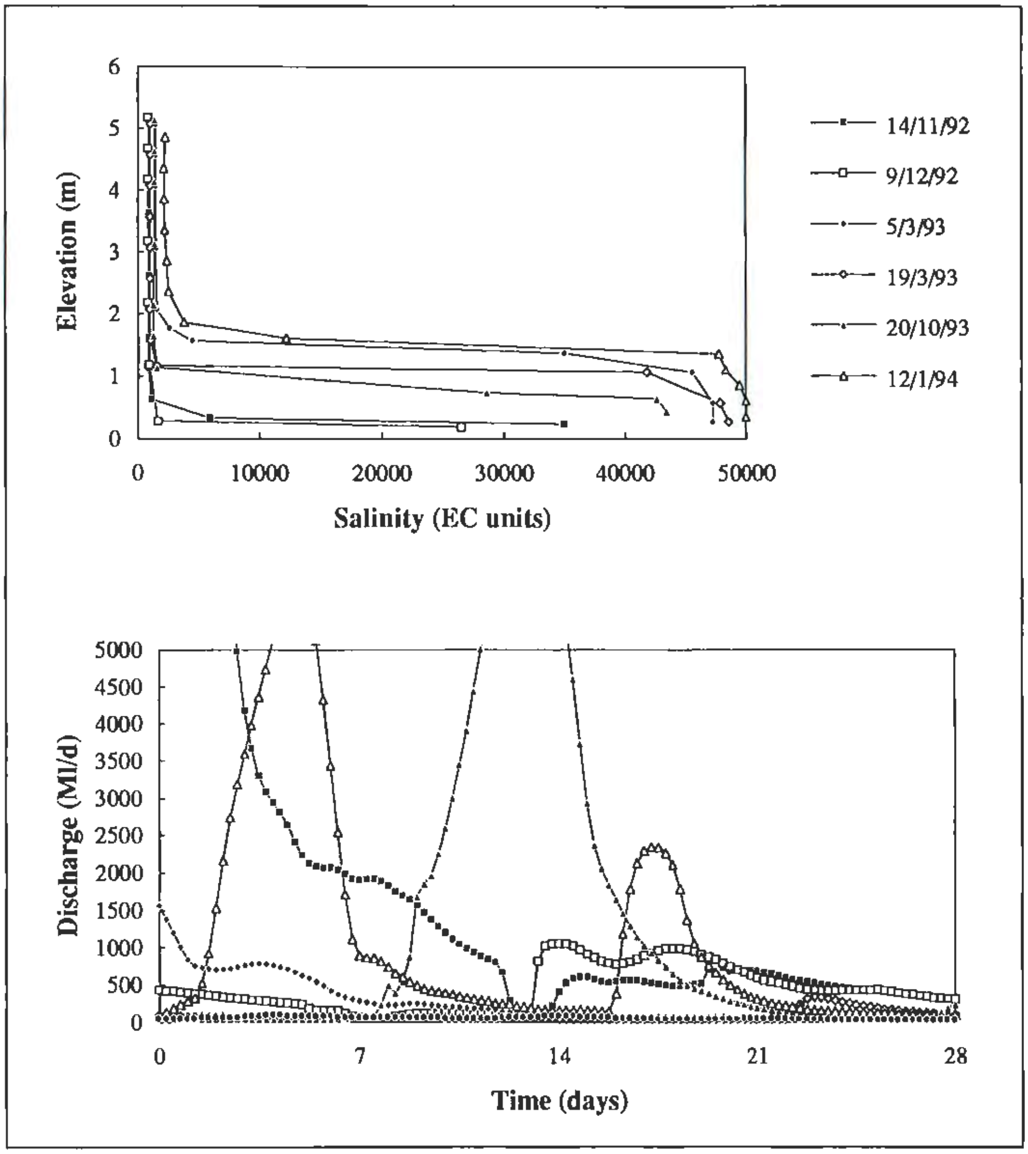


Figure 8.48: Vertical salinity profiles measured at Tarranyurk following the development of stratification and estimated discharge in the Wimmera River at Tarranyurk for the 28 days prior to the measurement of each profile.

significantly with relatively small changes in sensor elevation. For example, if the elevation of the sensor varied between gauge heights of 0.7 m and 0.9 m, the volume would vary from 20 m^3 to 70 m^3. Clearly given this factor and the small number of data points, the relationship between event size and formation rate observed in Table 8.1 may only be an artefact of random variation in probe elevation.

In the case of the manually measured profiles (Tables 8.2 to 8.5), one would expect a positive correlation between the equivalent groundwater volume and the time elapsed since the return to low flow conditions simply because the

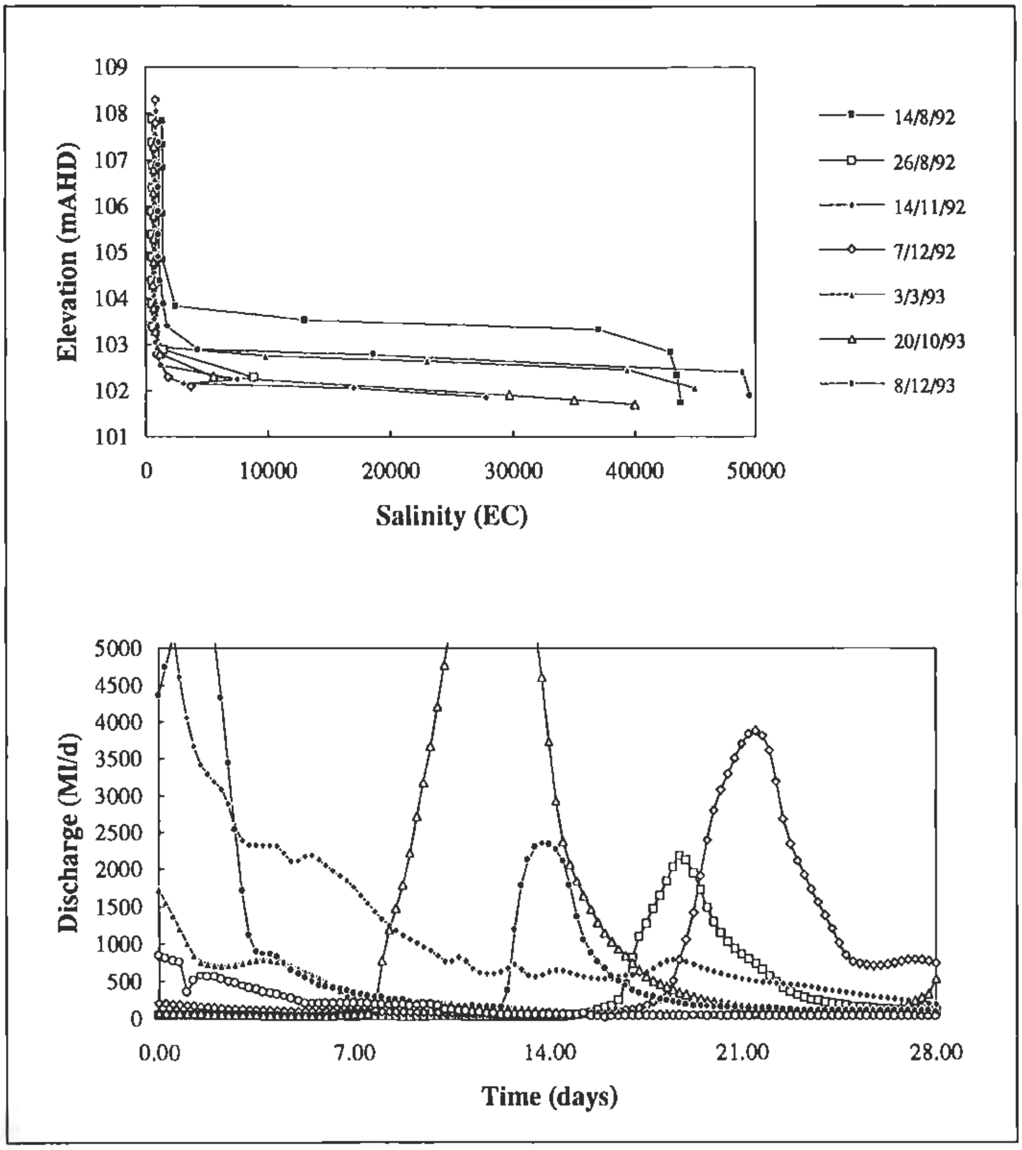


Figure 8.49: Vertical salinity profiles measured at Polkemmet following the development of stratification and estimated discharge in the Wimmera River at Tarranyurk for the 28 days prior to the measurement of each profile.

stratification is developing over time. Figure 8.50 shows that this is indeed the case; however there is a significant degree of scatter. Some of this scatter may result from poor vertical resolution of the profiles measured on some occasions, for example the 24/11/93 at Lower Norton 1. Enhanced mixing due to surface cooling may explain the observation on the 21/4/93 at Lower Norton 2.

Rates of accumulation of groundwater within a scour pool can be calculated from measured profiles in two ways. For the first profile measured after a mixing event the accumulated groundwater volume can simply be divided by T_{250}. This assumes that no significant mixing has occurred within this period and that formation began when the flow rate was 250 Ml/d. Where multiple profiles exist during the one stratification episode, the groundwater accumulation rate can be calculated on the basis of the difference in stored groundwater and time between individual profiles. The assumption that no significant mixing has occurred within this period is required for this estimation.

Time-weighted average formation rates were calculated from these data for each site. Points affected by partial mixing due to flow events, by mixing due to surface cooling or where saline water had overflowed from the scour hole were excluded. The average formation rates at Lower Norton 1, Lower Norton 2, Tarranyurk and Polkemmet were 0.010 Ml/d, 0.021 Ml/d, 0.008 Ml/d and 0.011 Ml/d respectively. If any mixing has occurred during periods used to calculate these rates, these time-weighted averages would underestimate the rate of groundwater inflow to the space below the halocline.

8.9.2.3 Formation Mechanism.

Inflows of groundwater leading to stratification may enter the scour depression through the bed of the depression or they may enter the stream channel at some other location and flow along the stream bed or down the stream banks and accumulate in the scour depression. Such a flow would be driven by buoyancy forces arising from the sloping channel bed or bank and the density difference between the river water and the groundwater and may result in some mixing between the river water and the groundwater. To investigate this issue an attempt was made to ascertain the location of groundwater inflows into Lower Norton 1 using seepage meters similar to those with which White and Denmead (1989) measured seepage from groundwater evaporation basins (Figure 8.51).

These devices consist of a large cylinder, closed at one end, which is pressed into the stream bed and collects any groundwater seeping into the stream at that location. Two tubes extend from the collection vessel to the surface. The first of these tubes is connected to a horizontal capillary tube located at the water surface. Any seepage into or out of the collection vessel leads to a flow in the capillary tube the velocity of which can be measured by observing the time of travel of the meniscus along the capillary tube. If seepage is into the stream the measurement is started with the capillary tube empty and when seepage is from the stream to the groundwater the measurement is started with tube full. The hydrostatic head in the collection vessel is maintained at the water surface due to the location of the capillary tube. The second tube is attached to the collection vessel at its highest point and collects any gas being evolved from the stream bed. It is necessary to

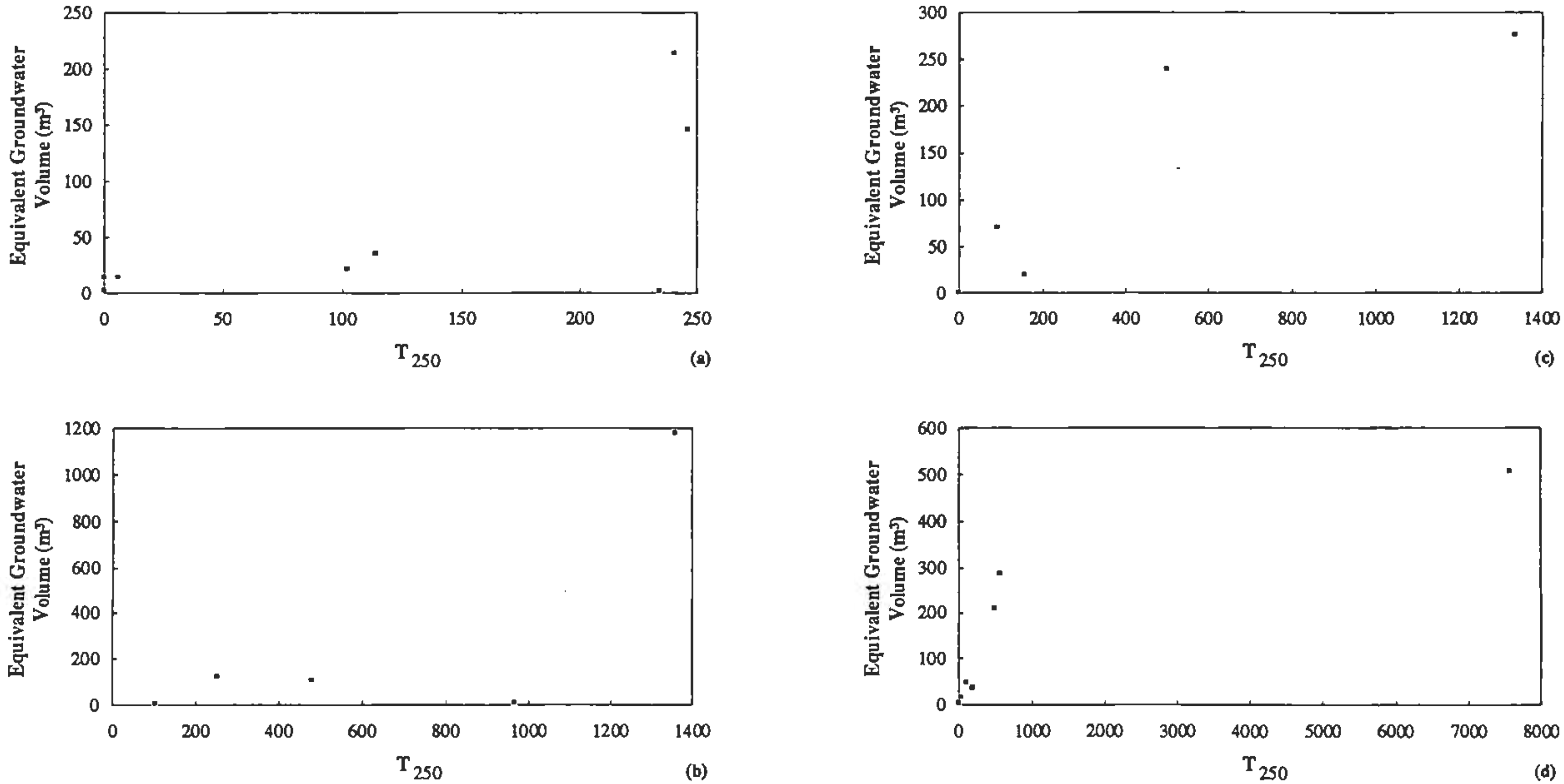


Figure 8.50: The variation of equivalent groundwater volume with T250 for: (a) Lower Norton 1; (b) Lower Norton 2; (c) Tarranyurk and (d) Polkemmet.

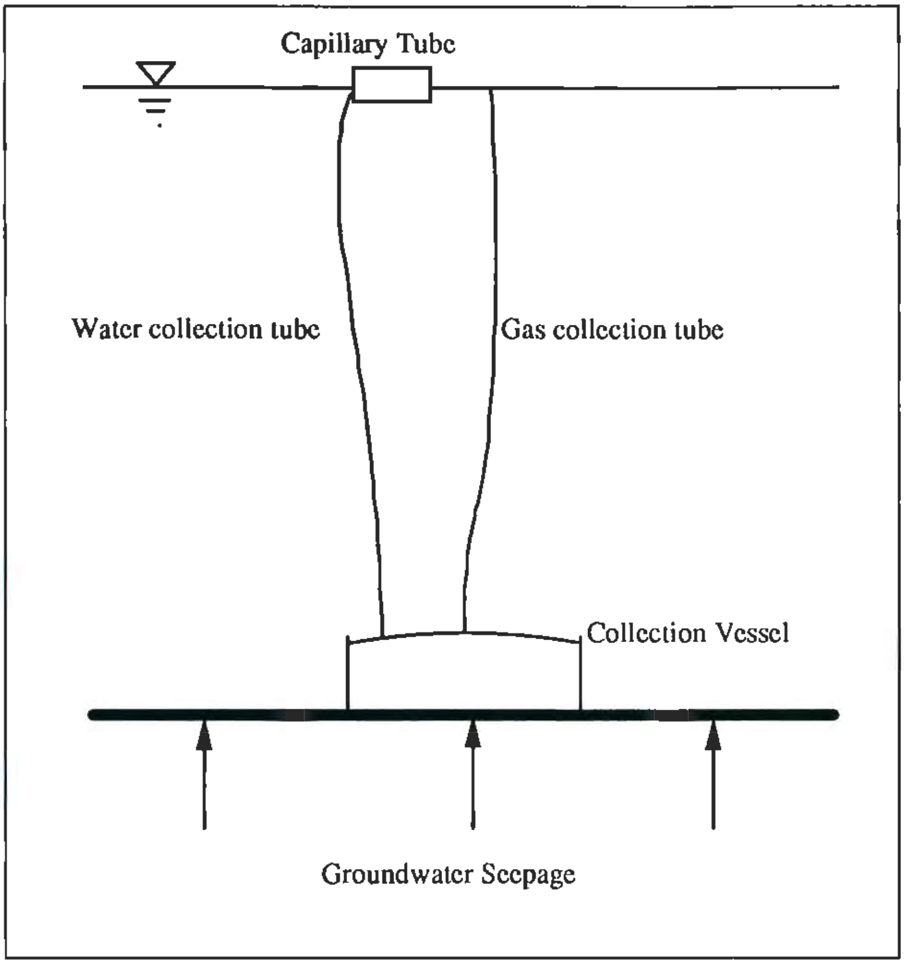


Figure 8.51: A seepage meter.

check that there is no evolution of gas from the stream bed as this would introduce an error into the measured seepage rate.

Six such meters were installed at regular intervals across the stream by a scuba diver. Difficulties were experienced in obtaining an adequate seal between the collection vessel and the stream bed and only three were successfully sealed. No seepage flows were detected using these three meters. White and Denmead (1989) claim that it is possible to detect seepage rates of 0.005 mm/d using this technique. The estimated volumetric rate of groundwater inflow for this site is 10 m^3/d. Given an area of approximately 5 000 m^2, this is equivalent to an areal average seepage rate of 2 mm/d which, given White and Denmead's (1989) experience, should be easily detectable. Two explanations are possible: the seepage meters failed in this case or there was no groundwater inflow at those points.

Groundwater seepage from the stream banks above the stream water level can be observed at Lower Norton 1. However the location of this seepage is very patchy. While this does not help to determine the location of groundwater inflows relative to the scour hole; it does indicate that groundwater seepage may be highly variable at small spatial-scales. Typical scales of the observed bank seepage are of the order of 10 m. If seepage of groundwater into the stream below the stream water surface was also highly spatially variable, it is quite probable that none of the three locations (each with a area of only 0.25 m^2 out of 5 000 m^2) would have significant groundwater inflows. A much larger number of locations would need to be used to clarify this issue. This was not practical because of the reliance on a diver to install the collecting vessels.

8.9.2.4 Modelling Saline Pool Formation.

From a modelling perspective the location of any groundwater inflow may not be important provided any groundwater entering above the halocline consistently flows into the scour hole or is consistently mixed into the upper layer. An assumption could then be made that a specific proportion of the groundwater entering the reach contributed to the water stored below the halocline. If this were the case, the rate of accumulation of groundwater should not depend on the volume of water stored under the halocline.

Figure 8.52 shows the relationship between formation rate and the equivalent groundwater volume stored. At Lower Norton 1 and Tarranyurk there is no relationship but at Lower Norton 2 and Polkemmet the formation rate increases with the volume stored. One explanation for these observations is the existence of a source of groundwater inflow from which the water diffuses into the upper layer during periods of limited stratification but which contributes to the water stored below the halocline during periods of extensive stratification. Alternatively saline water from a neighbouring scour depression that begins to overflow during periods of significant stratification may contribute to the stratification at these saline pools (see Figures 8.20 and 8.21).

Another explanation for the observations at Lower Norton 2 and Polkemmet is that some observations are affected by residual mixing. The low formation rates are associated with small volumes of stored saline water. These also generally coincide with the end of a flow event. The stream discharges are usually higher on these occasions and may be causing some mixing. This would then result in less stratification than expected given the assumption of zero mixing used in the above analysis.

It is interesting to note in the context of this discussion that prominent bank seepage was observed adjacent to scour pools which do not become stratified. River water salinities adjacent to such seeps were observed to be slightly higher than those for the water in the middle of the channel. This, along with the lack of stratification, indicates that the saline groundwater is simply diffusing into the main body of river water at these localities. It can therefore be concluded that in-flowing saline groundwater only leads to the development of stratification under certain conditions. However it is not possible to distinguish between the explanations provided for the behaviour at Lower Norton 2 and Polkemmet with the above analysis.

8.9.3 FORMATION BY BUOYANT STREAM FLOWS - BIG BEND.

The development of stratification at the Big Bend site can not be explained purely by direct intrusion of saline groundwater. The stratification observed during the autumn of 1992 is much more extensive than that observed during other low flow periods. Figure 8.53 shows the surface and bottom salinities at Big Bend and continuous and spot salinity measurements immediately upstream of Big Bend.

On two occasions during this low flow period the salinity of the river upstream of Big Bend exceeded 15 000 EC. The discharge on these two occasions was approximately 3 Ml/d. Given that only 4 Ml of saline water were stored in the Big Bend scour depression in the autumn of 1992, these highly saline stream flows could account for the observed stratification. However it is likely that groundwater intrusion also contributed to the observed stratification. This stratification episode is different to that observed by Anderson and Morison (1989c; 1989g) in July 1988 since it occurred during a period of very low flow. The event in July 1988 was the result of a relatively small (~700 Ml/d peak flow) event.

8.9.3.1 The Seasonal Behaviour of Big Bend Revisited.

High flows in the second half of 1992 and the summer of 1993 flushed the saline pool at Big Bend. During this period three stratification episodes were observed yet the salinity of stream water flowing into Big Bend was the same as the surface salinity at Big Bend. This implies that the stratification developed as a result of groundwater intrusion. Mixing at the end of the third of these stratification episodes was due to surface cooling. The water column continued to cool until June and no stratification was observed (Figure 8.18). Given that stratification appeared at the start of the low flow periods it is likely that groundwater was still

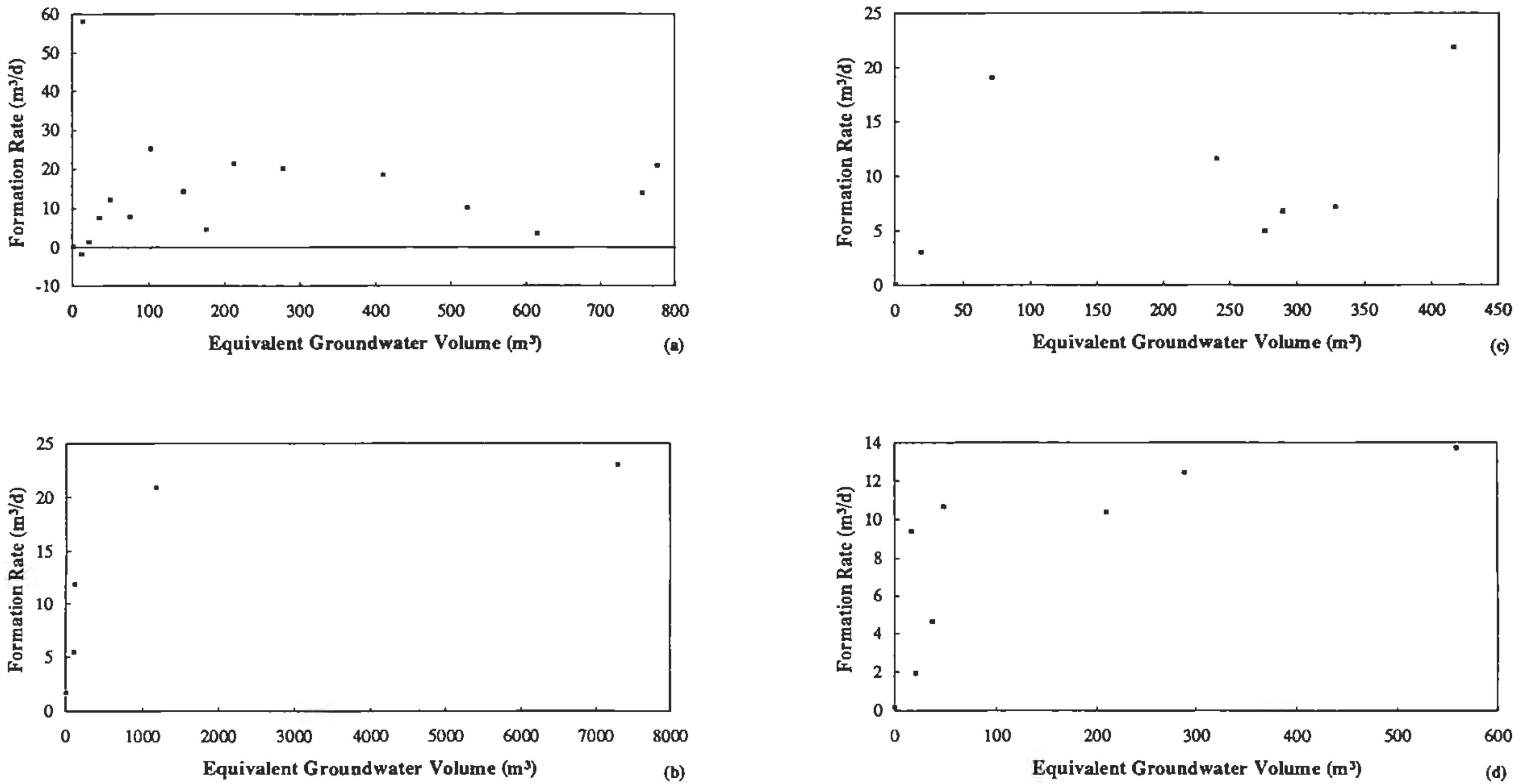


Figure 8.52: The variation in the rate of formation of stratification with the equivalent groundwater volume for: (a) Lower Norton 1; (b) Lower Norton 2; (c) Tarranyurk and (d) Polkemmet.

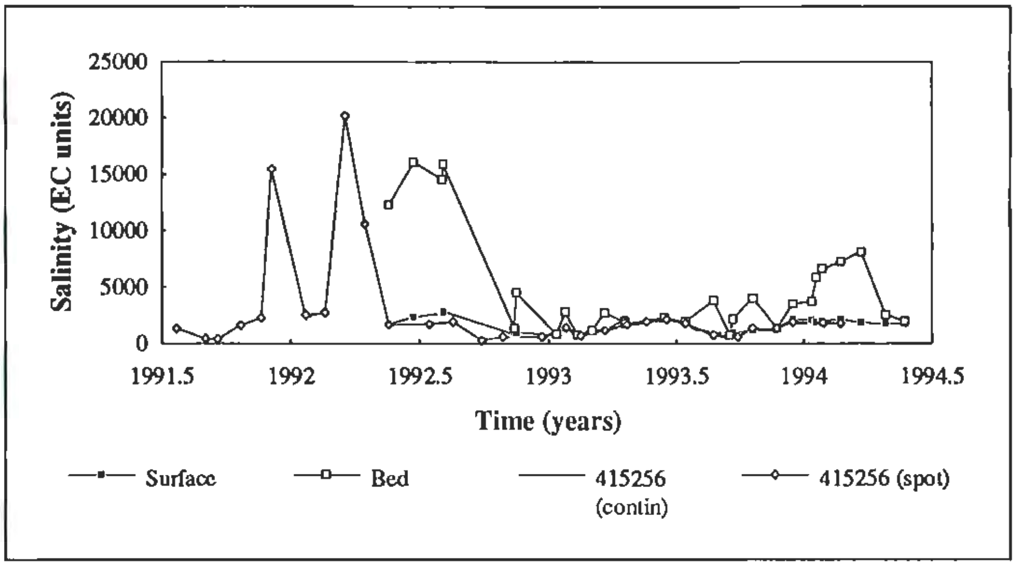


Figure 8.53: Variation in surface and bed salinity at Big Bend with time and variation in salinity upstream of Big Bend (415256) with time. Continuously monitored upstream salinities are provided where available and are supplemented with spot salinity samples.

seeping into the channel during this period and that the lack of stratification is a result of mixing due to the continued cooling of the water column.

During July and August 1993 the temperature of the water column increased slightly and stratification developed again. This stratification developed as a result of groundwater intrusion. The Big Bend saline pool was subsequently flushed twice and reformed during the spring of 1993. Stratification persisted through the summer of 1994 and was destroyed by mixing due to surface cooling in the autumn of 1994.

The seasonal behaviour of the saline pool at Big Bend can thus be explained when buoyant stream inflows and mixing due to surface cooling are considered. At Big Bend, stratification normally begins to develop at the onset of low flows and continues until a flow event flushes the saline pool. This basic pattern of behaviour is similar to that at Lower Norton, Polkemmet and Tarranyurk. However at Big Bend this behaviour is modified on some occasions by inflows of saline water from the river upstream and by mixing due to surface cooling.

8.10 SUMMARY.

The behaviour of saline pools at five different sites along the Wimmera River downstream of Horsham has been examined in this chapter. All of these saline pools are located on bends in parts of the river channel with permanent water that is relatively deep and wide. The sharpness of the bends varies significantly. The field observations have been augmented with results from a series of laboratory experiments performed by Nolan (1994).

The seasonal behaviour of four of the saline pools: Lower Norton 1 and 2, Tarranyurk and Polkemmet is primarily a function of the river discharge and saline groundwater intrusion. At these sites saline pools form at the onset of low flow conditions and the volume of saline water below the halocline increases with the duration of the low flow period. The saline pools are subsequently flushed when a high flow event occurs. This behaviour is typical of saline pools which form as a result of saline groundwater intrusion and which are primarily mixed by high stream flows.

The basic behaviour of the saline pool at Big Bend is similar to the other sites; however inflows of saline river water from upstream represent an additional formation process and surface cooling can cause additional mixing. These two additional processes lead to deviations from the basic relationship between discharge and stratification. The importance of saline river flows in the formation of other saline pools is unclear. Likewise, the importance of mixing due to surface cooling at other sites is unclear. However indirect evidence of enhanced mixing at Lower Norton 2 during a period of surface cooling was briefly discussed and the seasonal variation in the temperature of the water column indicates that surface cooling may be responsible for some mixing during the autumn of most years.

Longitudinal surveys were conducted in the Lower Norton and Polkemmet localities as part of this project. These surveys indicated that approximately half the scour depressions in each of these reaches were stratified at the time of the survey and that stratification affected approximately a quarter of the channel length in each area. Anderson and Morison's (1989c) surveys, while not directly comparable, also found that a significant number of scour depressions did not become stratified.

The formation of saline pools due to saline groundwater intrusion was examined in detail at the four sites dominated by saline groundwater intrusion. Saline pools begin to form during low but continuous flows that are generally between 200 Ml/d and 300 Ml/d. Time-weighted average formation rates were calculated for each of these sites and the relationship between the formation rate and the degree of stratification was examined. No relationship was found at Lower Norton 1 or Tarranyurk but at Lower Norton 2 and Polkemmet the formation rate increased as the stratification increased. This increase in the formation rate with the degree of stratification may have been due to the influence of residual mixing during the recession of a flow event. Alternatively it could be the result of saline groundwater infiltration into the channel only contributing to the stratification when that infiltration occurs below the halocline or contributions of saline water overflowing from neighbouring scour depressions. Details of the processes by which saline pools form from saline groundwater intrusion remain unknown.

Flushing of saline pools was examined in the field and the laboratory. The predominant mechanism leading to the flushing of a density stratified cavity with gently sloping end-walls involves a thin flow of saline water up the downstream slope of the depression. Mixing due to this process was characterised by Ri_{Ls}. While both the field and laboratory results could be explained using Ri_{Ls}; the mixing observed in the field was significantly greater than that observed in the laboratory. This can be partly explained by subtle differences in the velocity and length-scales used in the analysis. However it is believed that much of the difference between the field and laboratory observations could result from the effects of channel bends on mixing in the field. This hypothesis requires further testing through a series of laboratory and field experiments.

CHAPTER 9 - MODELLING OF SALINE POOLS.

9.1 INTRODUCTION.

A model of individual saline pools has been developed and applied to several saline pools in the Wimmera River. This model is referred to as Salipool. Salipool includes descriptions of saline pool formation due to groundwater intrusion and flushing due to buoyant overflows and is therefore applicable to saline pools dominated by these processes. The model does not include a description of mixing due to surface cooling nor formation of stratification due to buoyant river flows.

After Salipool has been described in detail, the methodology used to apply it to saline pools in the Wimmera River is discussed and a generalised calibration for the mixing component of the model is proposed. A sensitivity analysis is performed and the predictions of Salipool are used to explore the behaviour of saline pools in more detail. The model is then used to predict the effect of a very simple environmental flow management strategy on the saline pools at Lower Norton 1, Lower Norton 2, Polkemmet and Tarranyurk.

Using the experience gained with Salipool, a description of the behaviour of saline pools has been incorporated in the MIKE 11 model. The modified version of MIKE 11 is referred to as MIKE 11-SP. MIKE 11-SP is used to examine the effects of saline pools on the transport of salt in the Wimmera River.

9.2 THE SALIPOOL MODEL.

9.2.1 MATHEMATICAL MODEL.

For saline pools dominated by groundwater intrusion and mixing due to stream flows, a simple model can be constructed by representing the stratification with a layer of fresh water and a layer of saline water and solving the continuity equation for the volume of water stored in the saline layer, V_S. Groundwater seepage contributes to this layer at a rate equal to Q_{gw} and mixing leads to an outflow from the layer at the rate Q_{mix}. The continuity equation then becomes:

$$\frac{dV_S}{dt} = Q_{gw} - Q_{mix} . \qquad (9.1)$$

It is assumed that the mixing rate can be predicted using:

$$Q_{mix} = C_F.U_1.A_p.Ri_L^{-1} \qquad (9.2)$$

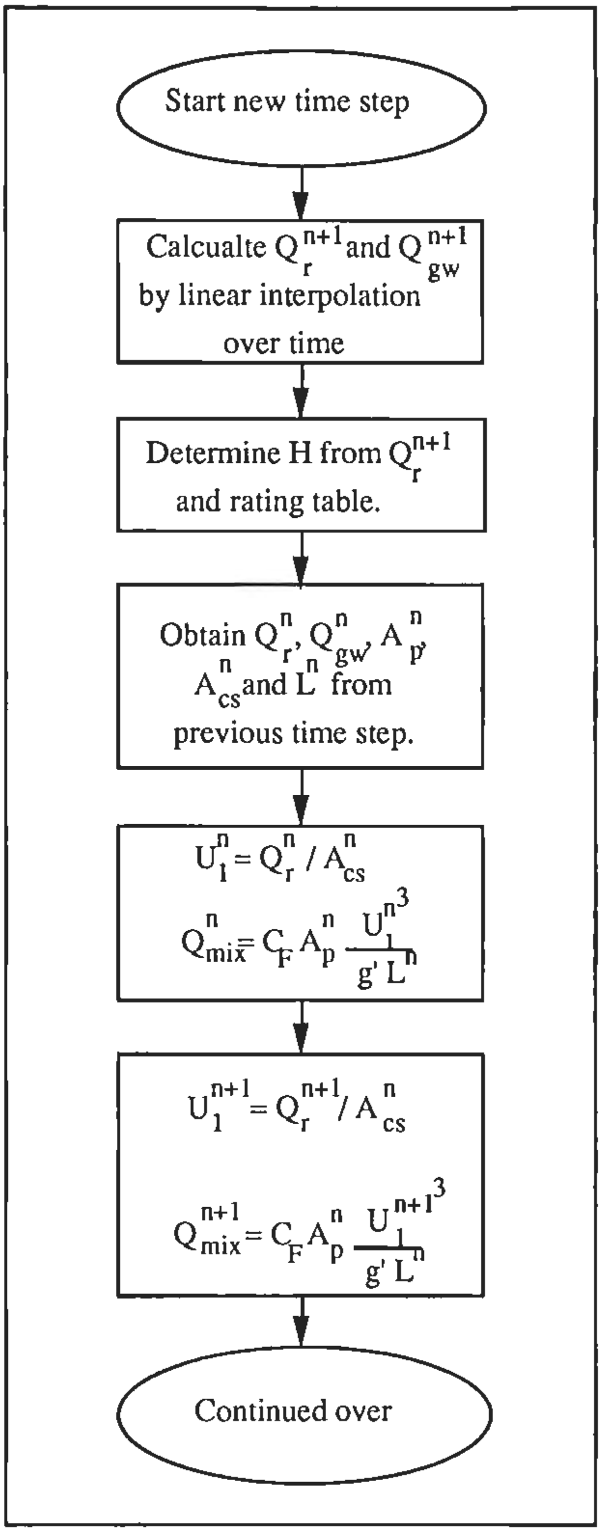


Figure 9.1: Flow chart for Salipool.

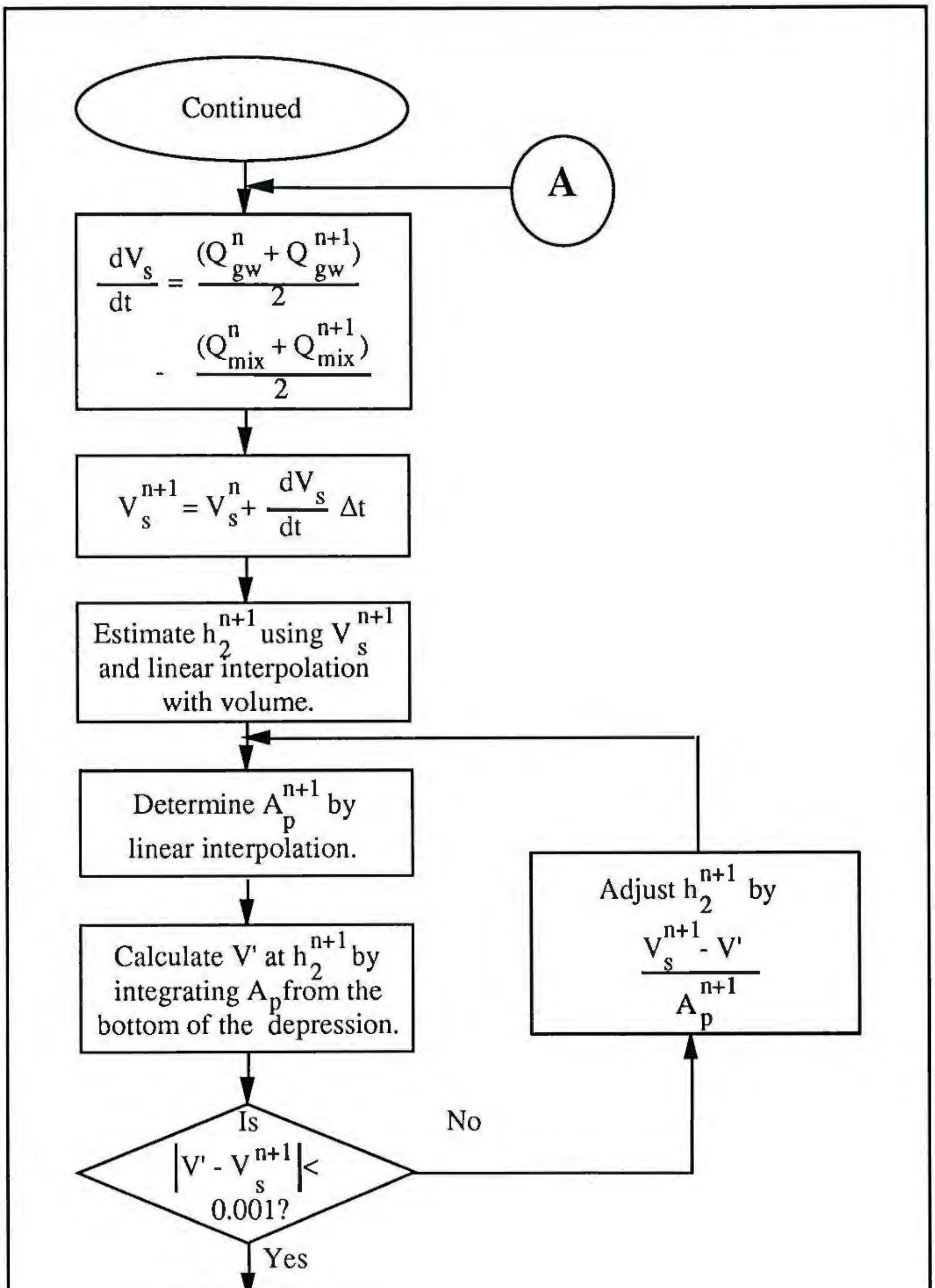


Figure 9.1: Flow chart for Salipool.

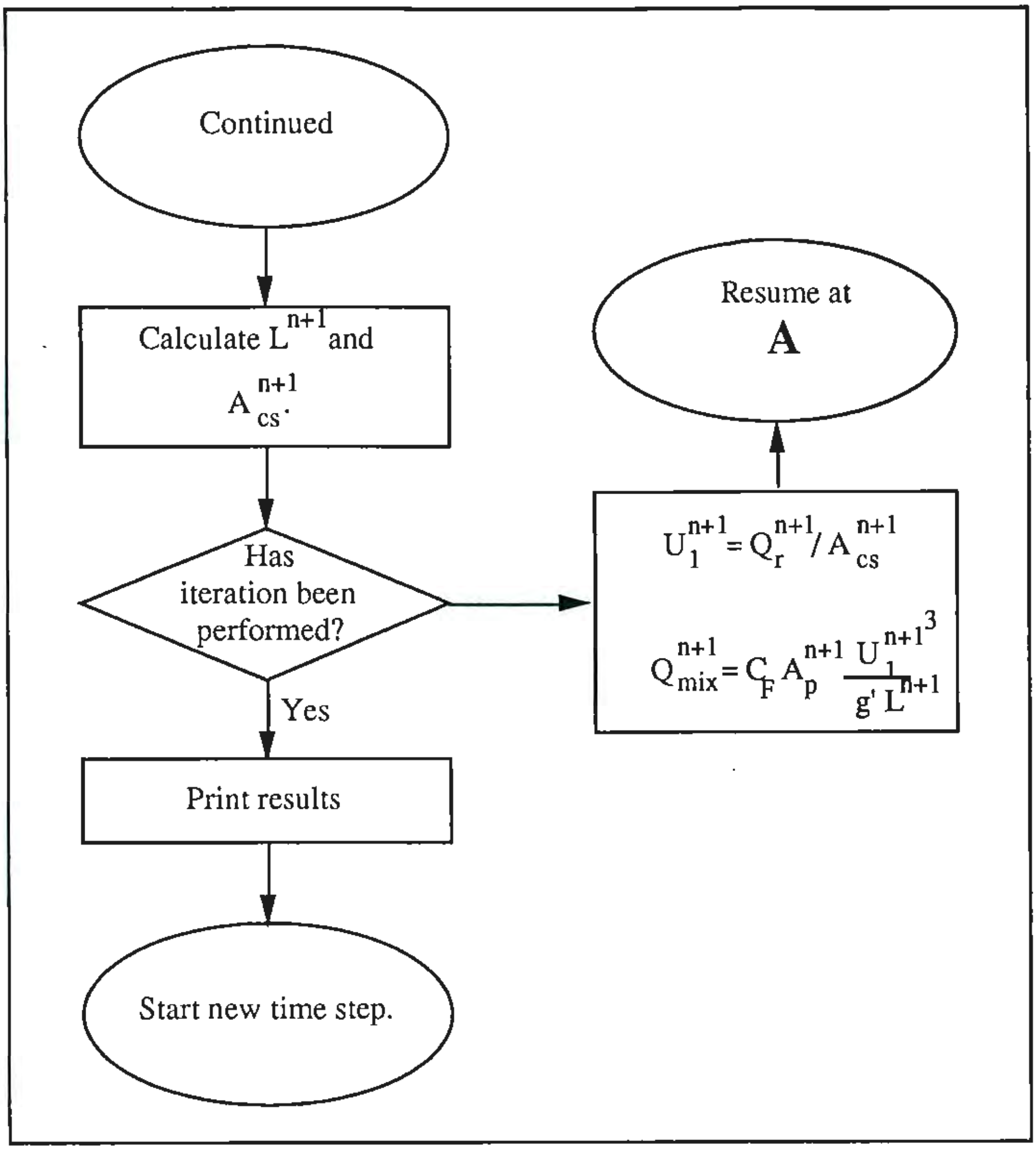


Figure 9.1: Flow chart for Salipool.

in which U_1 is the mean upper layer velocity, A_p is the plan area of the interface, $Ri_L = \frac{g' L}{U_1^2}$ is a Richardson Number based on the interface length and C_F is a flushing coefficient. Equation 9.2 is an empirical representation of observed flushing behaviour (see §8.7.2.3 and §8.7.3.3). If it is assumed that the rate at which groundwater contributes to the saline layer is independent of the amount of stratification, Q_{gw} can simply be specified as a time-series. Equation 9.1 can then be solved numerically provided the bathometry of the saline pool and the velocity of the upper layer are available.

9.2.2 NUMERICAL SOLUTION.

Equation 9.1 can be expressed in finite difference form:

$$\frac{V_s^{n+1} - V_s^n}{\Delta t} = Q_{gw}^{n+1/2} - Q_{mix}^{n+1/2}, \qquad (9.3)$$

where n refers to the initial time-step, n+1 to the subsequent time-step and n+1/2 indicates that an arithmetic average of the values at n and n+1 is taken. Since the length and plan area of the interface are required for the prediction of the mixing rate it is necessary to include the geometry of the scour hole in some way. It is convenient to use a table containing elevation, plan area and scour hole length. At the beginning of a simulation the volume of the scour hole at each elevation is calculated by integrating the plan area from the bottom of the scour depression to the required elevation using Equation 8.7.

Figure 9.1 shows details of the algorithms incorporated in Salipool and the main features of the calculations are described here. Given that Q_{mix} depends on the volume of saline water, Equation 9.3 is solved using an iteration. Q_{mix}^n, Q_{gw}^n and Q_{gw}^{n+1} can be calculated from the initial conditions and the time-series of stream and groundwater discharges supplied to the model. An initial guess for Q_{mix}^{n+1} is calculated by assuming V_s^{n+1} is equal to V_s^n for the purpose of calculating the geometric variables associated with the lower layer and using Q_r^{n+1} to determine the stage and U_1. An estimate of V_s^{n+1} is calculated and values of interface elevation, plan area and interface length are calculated for V_s^{n+1}. Then the upper layer velocity and Q_{mix}^{n+1} are re-calculated and the final estimate of V_s^{n+1} is calculated. If the value of V_s^{n+1} is negative the volume of saline water stored is set to zero. The program was written so that a maximum limit could be set on the volume of saline water stored. This allowed the simulation of saline pools from which the saline water can overflow to another scour hole such as at Lower Norton 1 and Polkemmet. This is discussed in more detail below.

During simulations the elevation of the interface is calculated using the volume of saline water and the table of scour hole geometry. An initial estimate of the interface elevation is calculated by linear interpolation with volume. The plan area, A_s^{n+1}, at that elevation is then calculated by linear interpolation with elevation and the volume, V', below that elevation is calculated by integrating the plan area between the base of the scour depression and that elevation. The interface elevation is then adjusted by an amount equal to $\frac{V_s - V'}{Ap'}$ and V' is recalculated. Iterations are performed until the error in V' is less than 0.001 m^3. This approach ensures consistency between the values of plan area and volume used in the calculations. The interface length is calculated by linear interpolation with elevation. A minimum of 0.1 m is placed on the interface length to ensure that Ri_L^{-1} is always defined.

9.2.3 MODEL TESTING.

A number of tests were conducted on Salipool. Simulations using different time-steps showed that results did not depend on the time-step used. Simulations with zero upper layer velocity and therefore zero mixing showed that the change in volume of saline water stored equalled the groundwater flow rate integrated over the simulation time as expected. A third test involved simulation of mixing in a depression with a constant plan area and length, a steady upper layer discharge per unit width, q, and zero groundwater inflow (Figure 9.2). An simple analytical solution for this problem can be derived as follows.

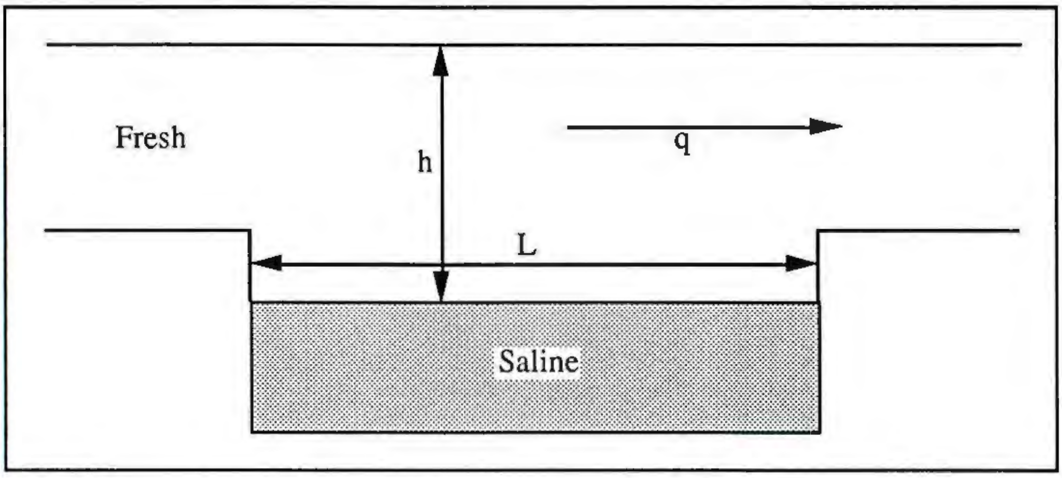


Figure 9.2: Geometry of depression used to derive analytic solution of Equation 9.1.

Since $Q_{gw} = 0$, Equation 9.1 can be simplified to:

$$\frac{dV_s}{dt} = - Q_{mix} \, . \qquad (9.4)$$

Substituting 9.2 into 9.4 and using $U_1 = q/h$ gives:

$$\frac{dV_s}{dt} = - \frac{C_F \, A_p \, q^3}{g' \, L \, h^3} \, . \qquad (9.5)$$

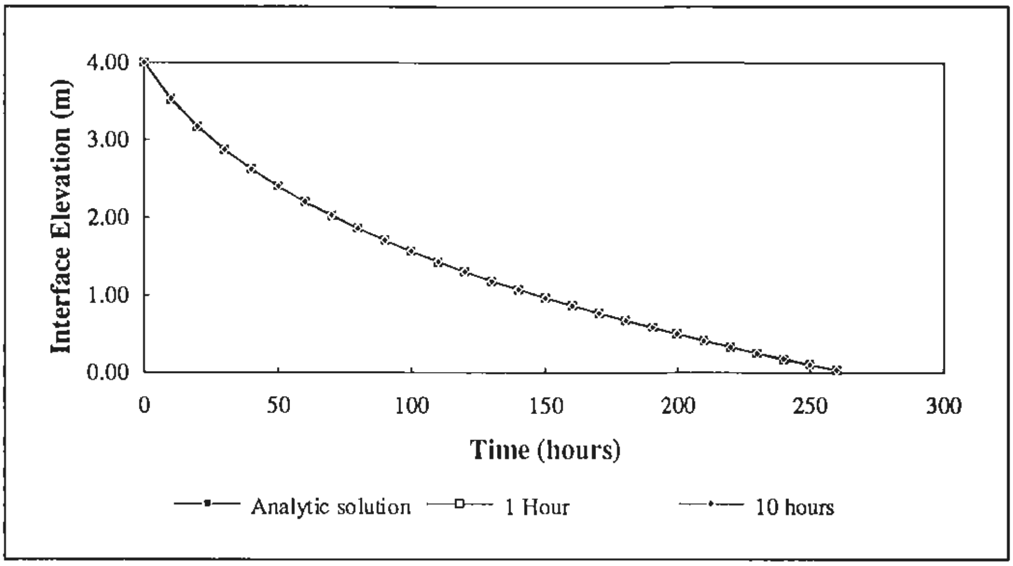


Figure 9.3: Comparison of analytic and numerical solutions for flushing of a cavity with a rectangular plan form 25 m long and 4 m wide and a depth of 4 m. The flow rate was 1 m3/s and the water surface was 8 m above the bottom of the cavity. A salinity difference of 15 000 EC units was used in the calculations.

From the geometry it is clear that:

$$\frac{dV_s}{dt} = - A_p \frac{dh}{dt} . \tag{9.6}$$

Equating 9.5 and 9.6 and integrating subject to the initial condition that $h = h_0$ at t = 0 gives:

$$h = \sqrt[4]{\frac{4 C_F q^3}{g' L} + h_0^4} . \tag{9.7}$$

Figure 9.3 shows the results of two simulations for a rectangular cavity 4 m wide and 25 m long conducted with time-steps of 1 hour and 10 hours. The discharge was 1 m^3/s, the water surface was 8 m above the bottom of the depression and the initial interface height was 4 m above the bottom of the depression. The analytical solution for this case is also shown and there is good agreement between the three solutions.

9.2.4 INTERPRETATION AND DATA REQUIREMENTS.

The most significant assumption made in Salipool is that a saline pool can be represented by a layer of saline water and a layer of fresh water. It has been assumed in the applications of the model described below that the salinity of the lower layer is the same as the salinity of the groundwater. The volume of the saline layer predicted by the model must therefore be interpreted as the equivalent groundwater volume stored below the halocline and not the total volume of water affected by stratification. Other assumptions made in the model are: the saline pool forms as a result of direct groundwater intrusion; the rate of formation of stratification is independent of the amount of stratification present; mixing is due solely to river flows and mixing by river flows can be represented by Equation 9.2.

Data required for Salipool can be obtained from simple field measurements; however a series of vertical salinity profiles are required which must be collected over a sufficiently long period. The geometry of the scour depression and a flow rating curve are required. In the following applications the scour depression geometry was obtained from channel surveys. The rating curves for different sites were constructed from stream gaugings for Lower Norton 1 and 2, from observed discharge immediately upstream of and stage at Big Bend and from observed stage and estimated flows at Tarranyurk and Polkemmet. Rating curves only need to be accurate for the range of discharges up to the that at which mixing is complete. The saline layer disappears at higher flows and thus the velocity of the upper layer is unimportant.

Discharge-time-series are required for both the stream and the groundwater entering the stream. In the applications described, stream discharge was obtained from a nearby stream gauging station and the groundwater flow rate was assumed to be constant and equal to the time-weighted average discharge observed during formation of stratification (see Chapter 8.9.2.2). A series of vertical salinity profiles obtained during low flow periods is necessary to obtain this information. An appropriate value of C_F is required and was obtained by calibration as described in the next section. An approximate generalisation of C_F is provided which could be used for future applications; however this generalisation requires further testing and a decrease in the accuracy of simulations would be expected. Detailed monitoring of mixing events provides the best information for this calibration, however continuous monitoring of bottom salinity and profiles measured shortly before the mixing event can be used.

9.3 APPLICATION OF SALIPOOL TO THE WIMMERA RIVER.

9.3.1 CALIBRATION OF THE MIXING COEFFICIENT.

To obtain a value of C_F from observed data at Lower Norton 2, Big Bend and Tarranyurk a slightly modified version of the above model was constructed and applied to mixing events at these sites. This modification affects the way in which the density difference is calculated. A vertical salinity profile was specified at the start of the simulation and it was assumed that the salinity below the halocline was equal to the salinity in the specified profile at the elevation of the halocline. This was calculated by linear interpolation with elevation and used to determine g'. The modified model is referred to as Salipool-Pr and it allows the effects of a non-uniform lower layer salinity profile on the mixing to be incorporated provided that the initial salinity profile is known. Plan areas and lengths of scour holes and cross-sectional areas at the deepest part of the scour hole were measured at different elevations and supplied to the model.

Salipool-Pr was initially applied to the two events for which detailed field data had been collected at Lower Norton 2 on the 15-16/8/92 and at Tarranyurk on the 3/9/93. The coefficient C_F in Equation 9.2 was calibrated to reproduce the observed behaviour at these two sites. Figures 9.4 and 9.5 show the results of these two simulations. Salipool-Pr was then applied to a mixing event at Big Bend for which vertical salinity profiles prior to the start of the event (4/8/92) and during the final stages of mixing (1620 hours, 16/8/92) were available (Figure 9.6). The flow event that resulted in the observed mixing was simulated and the salinity profile measured on the 4/8/92 was used to specify the salinity profile at the start of the event. Figure 9.7 shows the simulated interface elevation and the single observation.

Events at Lower Norton 1 on the 28/1/93 and the 11/3/93 which were observed by the continuous salinity recorder located 0.5 m above the bottom of the scour depression were used to calibrated C_F at Lower Norton 1. Salinity profiles measured on the 25/1/93 and the 3/3/93 were available; however due to the rapid rate of formation at Lower Norton 1 these are not a true representation of the conditions at the start of the subsequent events. Therefore Salipool was used and the time-weighted average formation rate at Lower Norton 1 was used to specify Q_{gw}. Simulations were conducted for the periods between the profile measurement and the flushing of the pool. Figures 9.8 and 9.9 show the simulations and the volume of saline water stored below the halocline when the

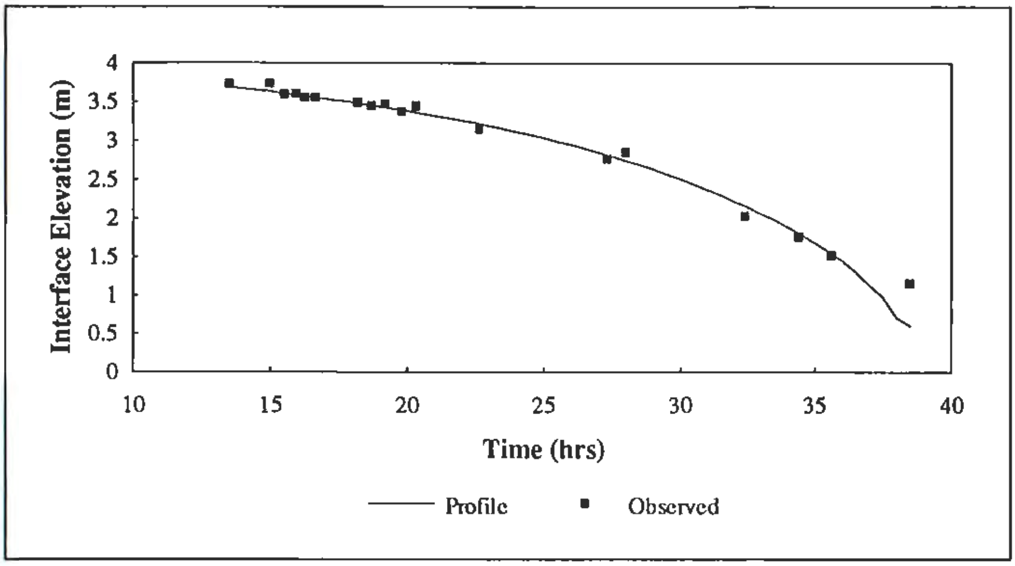


Figure 9.4: Simulated and observed mixing at Lower Norton 2 on the 15/8/92 and the 16/8/92. Simulation uses Salipool-Pr with $C_F = 0.45$. Time is from 0000 hours on the 15/8/92.

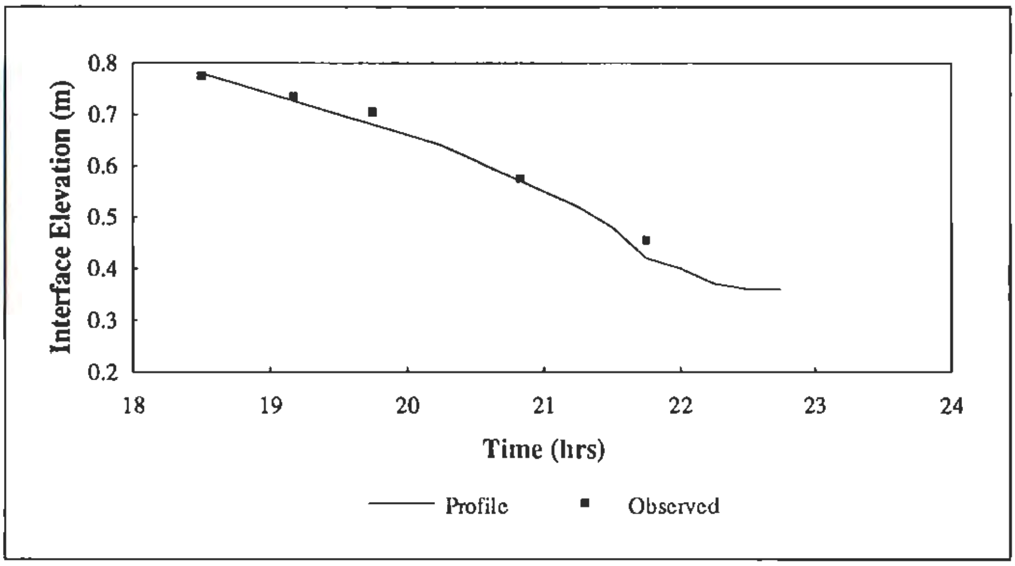


Figure 9.5: Simulated and observed mixing at Tarranyurk on the 3/9/93. Simulation uses Salipool-Pr with $C_F = 0.20$. Time is from 0000 hours on the 3/9/93.

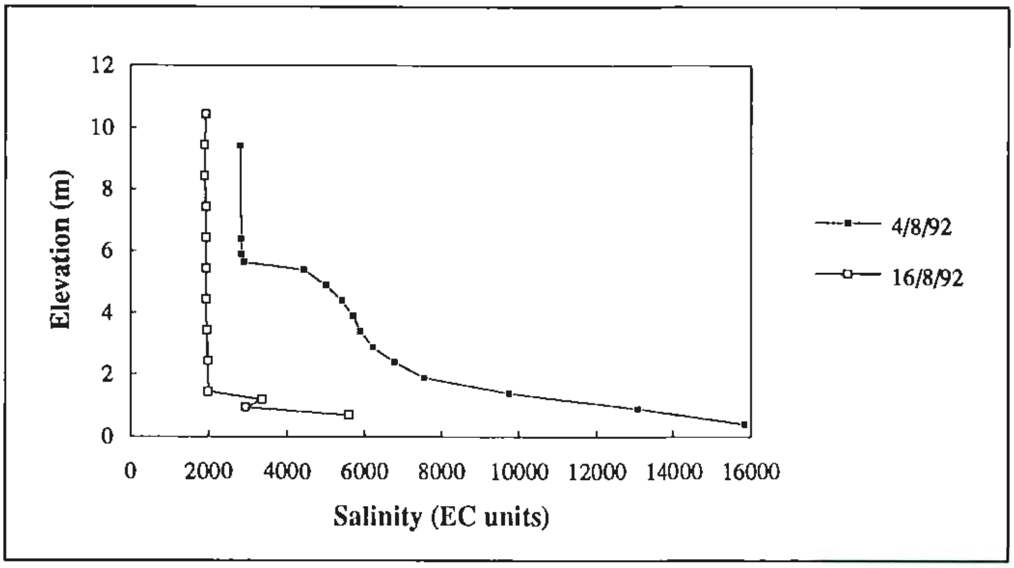


Figure 9.6: Vertical salinity profiles measured at Big Bend at 1345 hours on the 4/8/92 and at 1620 hours on the 16/8/92.

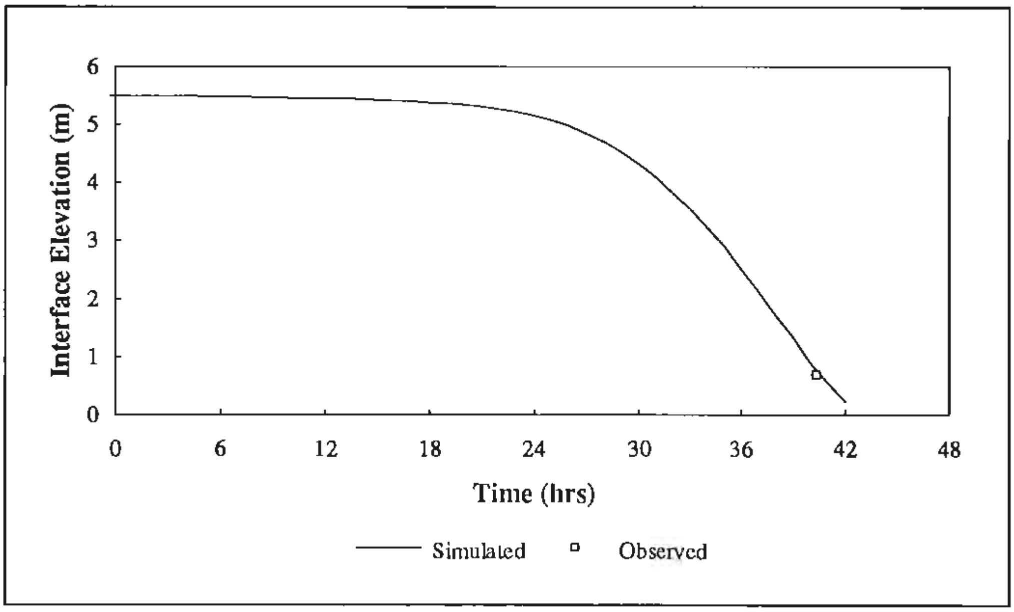


Figure 9.7: Interface elevation simulated with Salipool-Pr during mixing at Big Bend on the 15/8/92 and the 16/8/92 and an observed interface elevation during the last stages of mixing. Time is from 0000 hours on the 15/8/92.

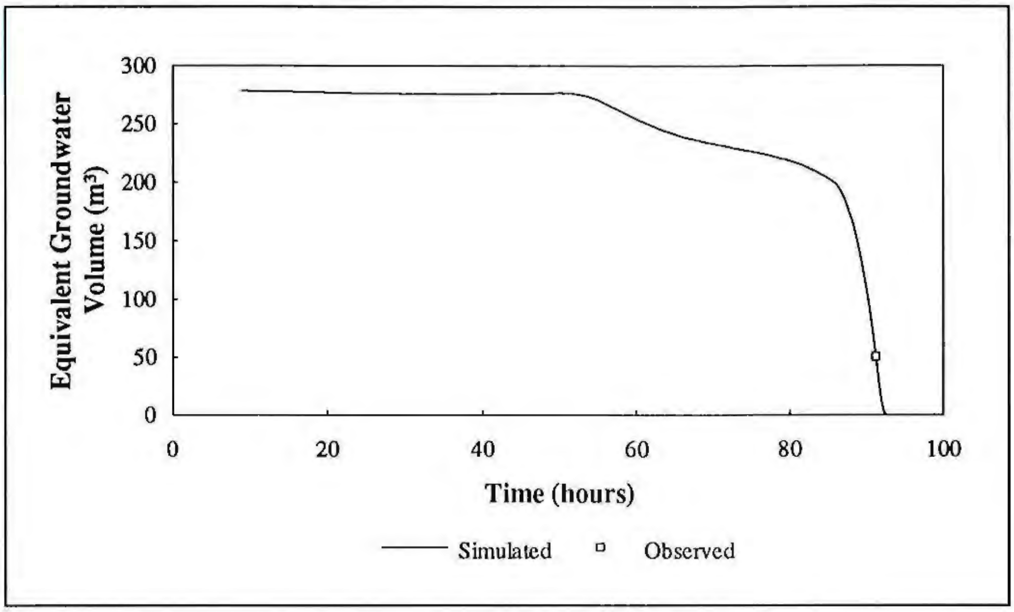


Figure 9.8: Simulated saline water storage at Lower Norton 1 between the 25/1/93 and the 28/1/93 and observed saline water storage during the last stages of mixing. $C_F = 0.19$ and time is from 0000 hours on the 25/1/93.

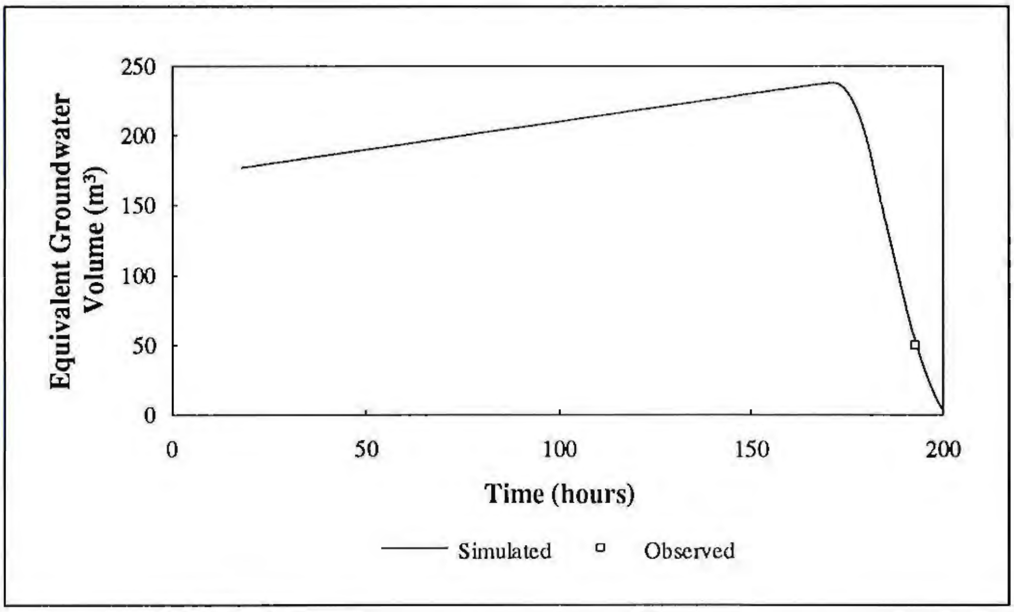


Figure 9.9: Simulated saline water storage at Lower Norton 1 between the 3/3/93 and the 11/3/93 and observed saline water storage during the last stages of mixing. $C_F = 0.22$ and time is from 0000 hours on the 3/3/93.

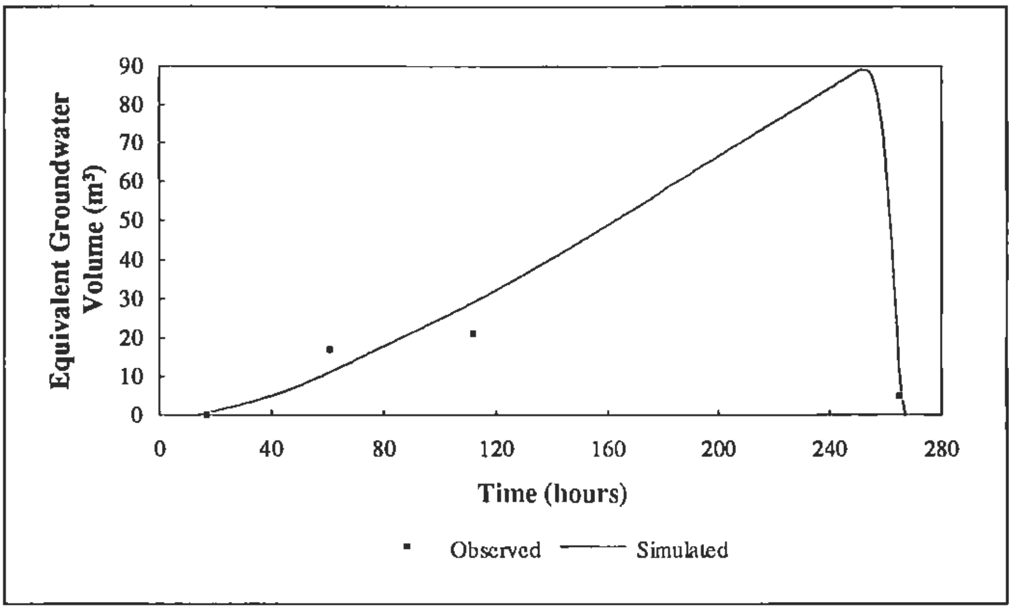


Figure 9.10: Simulated and observed saline water storage at Polkemmet from 12/11/92 to the 23/11/92. The time is from 0000 hours on the 12/11/92.

interface moved below the continuous salinity sensor. A similar procedure was adopted for the calibration of C_F for Polkemmet where continuous data had been collected by Kaiela Fisheries Research Station (T. Ryan, KFRS, Per. Com.) for an event on the 23/11/92 and profiles were available for the 12/11/93, 14/11/93 and 16/11/93 (Figure 9.10).

It is noted that the use of Salipool-Pr in the calibration of C_F at some sites implies that the assumptions used to calibrate C_F are slightly different to those made in the Salipool model. Since the Salipool would normally be used, simulations using a homogenous saline layer with salinity equal to the groundwater salinity were conducted for the events at Lower Norton 2 and Tarranyurk. Figures 9.11 and 9.12 show the results of these simulations. It is noted that the calibrated value of C_F is also applicable when the two layer assumption is made.

This can be explained as follows. Using the groundwater salinity and the equivalent groundwater volume in the simulations as opposed to the known salinity profile means that the density difference is larger. This tends to reduce the mixing rate. On the other hand the equivalent groundwater volume is smaller than the total volume of saline water thus a smaller mixing rate is required to model the observed behaviour. There is also a reduction in the length of the interface which

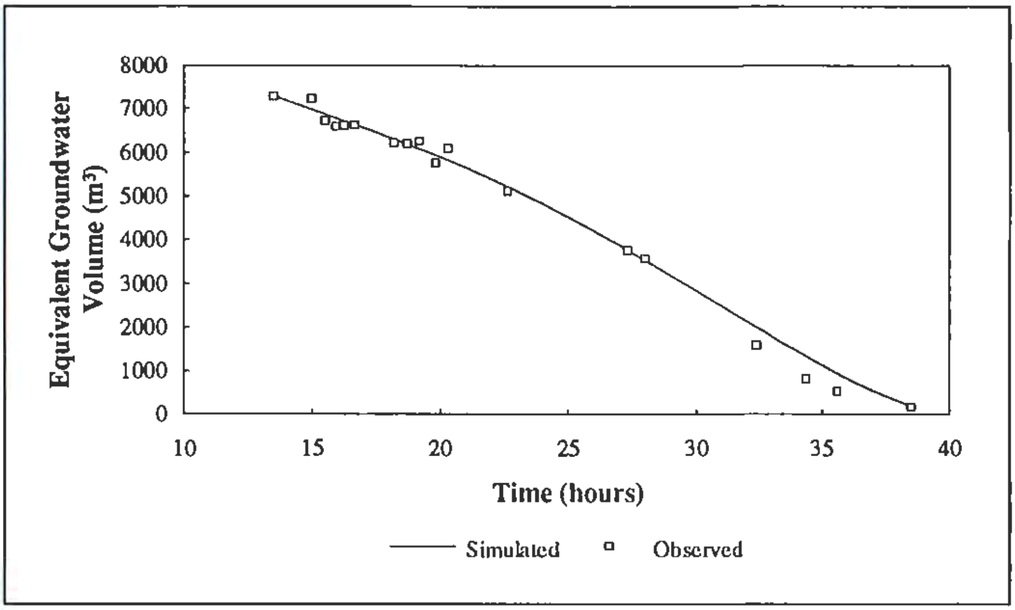


Figure 9.11: Simulated and observed mixing at Lower Norton 2 on the 15/8/92 and the 16/8/92. Simulation uses Salipool with $C_F = 0.45$. Time is from 0000 hours on the 15/8/92.

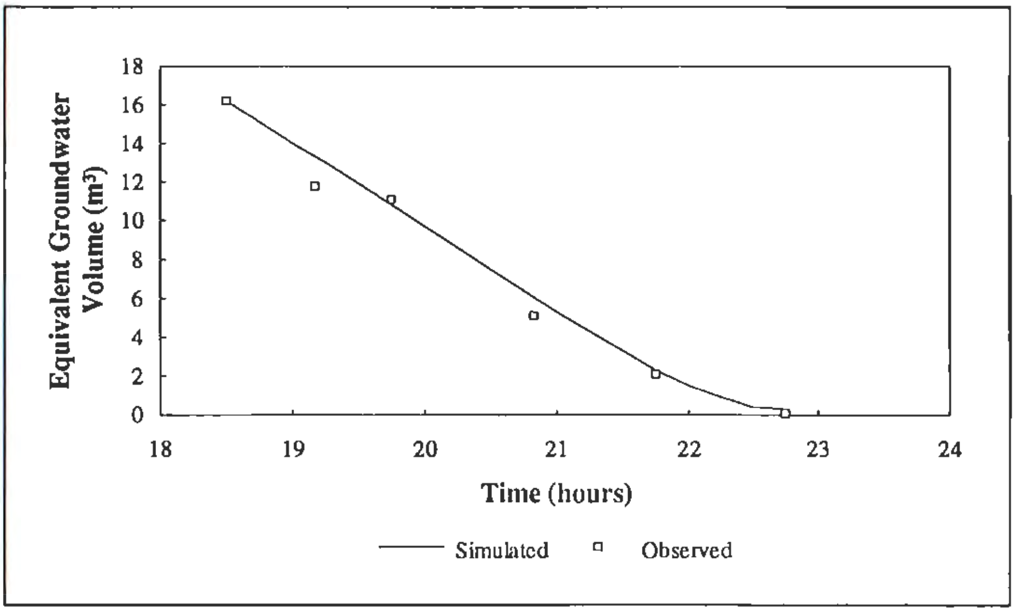


Figure 9.12: Simulated and observed mixing at Tarranyurk on the 3/9/93. Simulation uses Salipool with $C_F = 0.20$. Time is from 0000 hours on the 3/9/93.

tends to increase the predicted mixing rate and a reduction in the interfacial area which reduces the predicted mixing rate.

Site	C_F	$\tan\theta$	A/A_D	C_F'	D/R_b
Lower Norton 1	0.15	0.038	1.2	0.0033	0.048
Lower Norton 2	0.45	0.018	1.2	0.0047	0.036
Big Bend	0.90	0.164	1.5	0.043	0.156
Tarranyurk	0.20	0.035	1.03	0.0064	0.037
Polkemmet	0.60	0.020	1	0.012	0.086
Laboratory	0.0080	0.087	1	0.0007	0

Table 9.1: Calibrated and adjusted values of mixing coefficient.

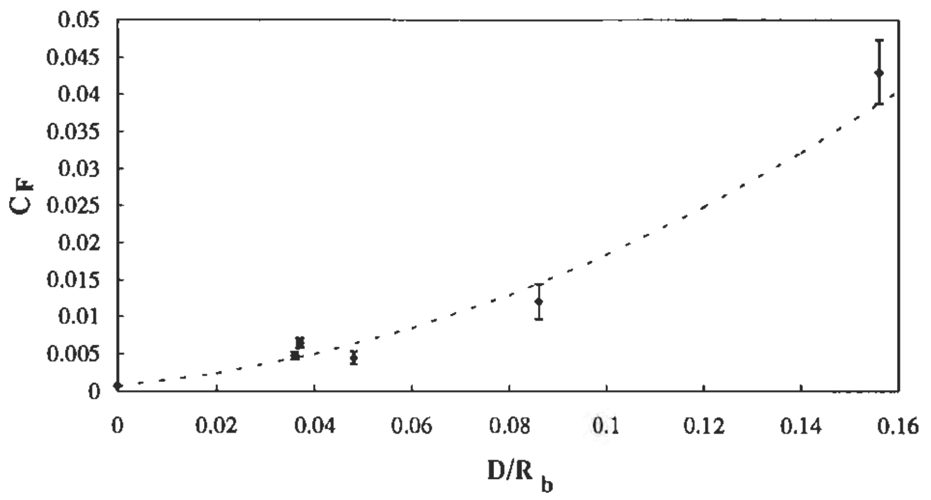


Figure 9.13: Relationship between mixing coefficient C_F' and bend sharpness D/R_b.

9.3.1.1 Generalisation of the Mixing Coefficient, C_F.

The calibrated values of C_F are not directly comparable between sites. As argued in Chapter 8 the appropriate Richardson Number for scaling the mixing is $Ri_{Ls} = \dfrac{\tan\theta\, g'\, L}{U_{1D}^2}$. The velocity used in the model, U_1, is calculated at the deepest point along the scour hole and no allowance is made for the downstream slope of the scour pool. It is possible to adjust C_F to account for the change in velocity along the scour depression and for the downstream slope of the scour depression. Table 9.1 shows the calibrated values of C_F the bed slopes and ratio of cross-sectional area at the deepest point in the scour pool, A, and at the point where the downstream extremity of the stratification would typically have been located during the mixing event, A_D. The adjusted values of mixing coefficient, C_F', and the ratio of the mean channel depth, D, and bend radius, R_b are also included.

Figure 9.13 shows the relationship between D/R_b and C_F'. The estimated errors are indicated. D/R_b is a measure of bend sharpness and Figure 9.13 shows that the mixing coefficient increases with bend sharpness. The observed relationship between the mixing coefficient and bend sharpness is consistent with the arguments presented in Chapter 8.7.4. It appears that the bend sharpness provides a reasonable basis for generalising the mixing coefficient; although significant errors would be expected due to the limited number of data points currently available. A provisional relationship between D/R_b and C_F' is shown.

9.3.2 LONGER TERM MODELLING OF SALINE POOLS.

Salipool was applied to the saline pools at Lower Norton 1, Lower Norton 2, Polkemmet and Tarranyurk for the period over which salinity profiles have been collected at these sites. The calibrated values of C_F were used in the mixing relationship and the time-weighted mean formation rate calculated in Chapter 8 was used for the groundwater inflow rate. Figures 9.14, 9.15, 9.16 and 9.17 show the results of these simulations and the observed groundwater storage on occasions when vertical salinity profiles were measured. Because the observed data were used to estimate the groundwater inflow rate these simulations do not represent a truly independent test of the model. Salipool was not applied to Big Bend for this longer period due to the significant effects of surface cooling on mixing and saline river inflows on the development of stratification at this site.

At Lower Norton 1 the presence or absence of stratification is well predicted and the amount of stratification present is generally predicted to within an error of -50% and +100%. Salipool over-predicts the amount of stratification during the middle part of 1993 at Lower Norton 1. This is partly due to the fact that the scour depression at Lower Norton 1 fills and overflows during extended low flow periods. The calculated equivalent groundwater volume (see §8.9.2.1 for details) only accounts for saline water within the scour depression. Because some of the water in the scour depression will be less saline than the assumed groundwater salinity, the equivalent groundwater volume will be less than the total volume of the depression even though the depression is completely full of saline water. However Salipool assumes that all the water in the depression has a salinity equal to the groundwater salinity. Thus neglecting saline water that has overflowed from the scour depression can account for some of the observed error.

Data for testing the predicted stratification at Lower Norton 2 were limited to only seven profiles which were predominantly collected during periods of little or no stratification. Nevertheless the observed stratification was generally well

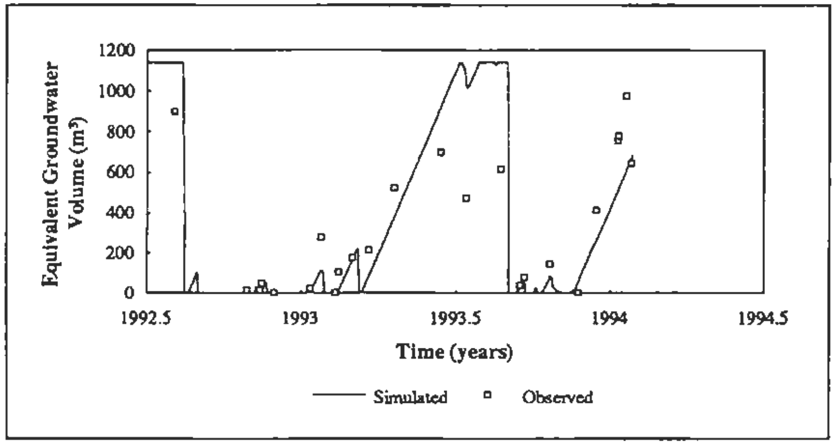


Figure 9.14: Simulated and observed stratification at Lower Norton 1 for the period 1/7/92 to 28/1/94.

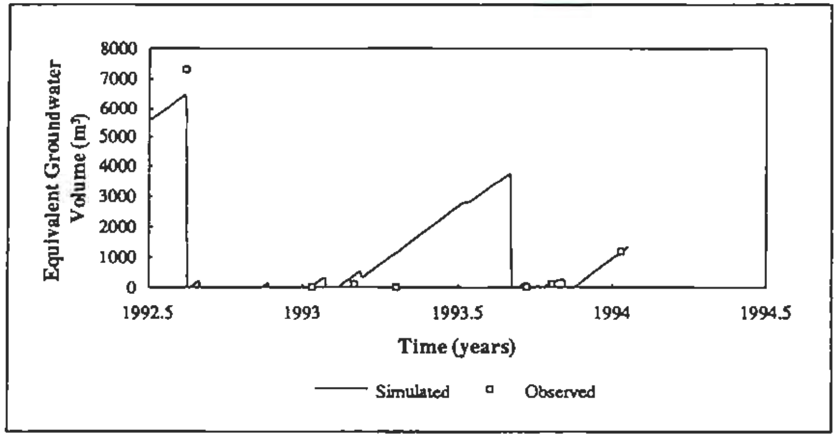


Figure 9.15: Simulated and observed stratification at Lower Norton 2 for the period 1/7/92 to 28/1/94.

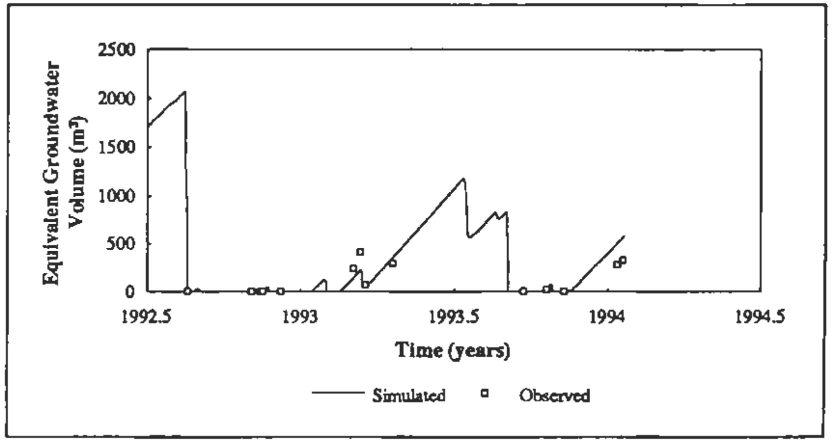


Figure 9.16: Simulated and observed stratification at Tarranyurk for the period 1/7/92 to 20/1/94.

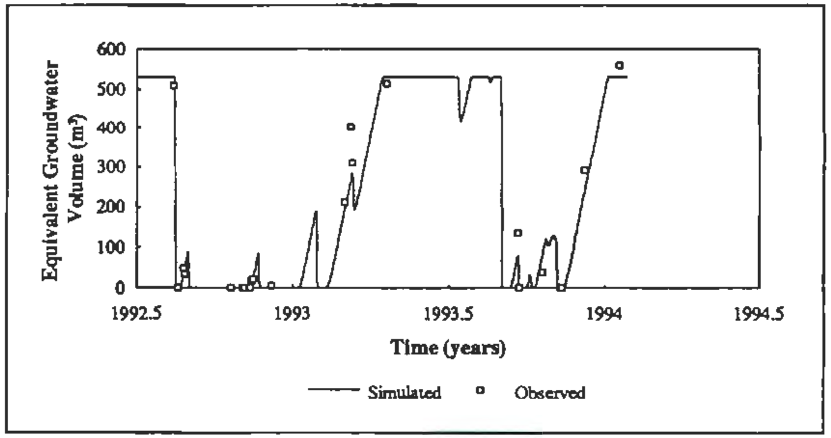


Figure 9.17: Simulated and observed stratification at Polkemmet for the period 1/7/92 to 27/1/94.

predicted with the exception of the third observation in 1993 when stratification was significantly over-predicted. This indicates that either there was an additional source of mixing during the period before the observation or the groundwater inflow rate was significantly less than normal. It is noted that significant stratification had developed at Lower Norton 1 during this period which indicates that groundwater was still flowing into the stream. Mixing associated with surface cooling was observed at Big Bend during this period and it is possible that such mixing, which is not included in Salipool, is responsible for the error in the prediction.

Predictions of stratification at Polkemmet are good; however it is noted that this scour depression fills rapidly and would then begin to interact with neighbouring scour depressions. Salipool does not describe such interactions between neighbouring scour depressions and so the predictions during periods when the scour depression is full can only indicate that extensive stratification exists. It would be possible to simulate interactions between neighbouring scour depressions relatively easily by letting saline water spill from one to the other. This has not been done because testing of the model would require salinity profiles from all the scour depressions included in the simulations and these are not currently available. At Tarranyurk stratification predictions are good during periods with a small amount of stratification. Additional data is required to test the model during periods of extensive stratification at Tarranyurk.

The above simulations indicate that Salipool is capable of predicting the pattern of fluctuations in stratification at saline pools in the Wimmera River provided the behaviour of these pools is dominated by groundwater intrusion and mixing due to buoyant overflows. However errors of up to -50% and +100% in predicted stratification can occur. Sources of error include: incorrect specification of flow rates; interactions between neighbouring scour depressions when the depressions are full of saline water and errors in measurement of the salinity profiles. Errors also occur when the model assumptions are violated, for example when surface cooling leads to significant mixing or if groundwater inflow rates were constant over time.

9.3.2.1 Seasonal Behaviour of Saline Pools.

The results from the above simulations give some insight into the seasonal behaviour of saline pools in the Wimmera River. Firstly the time-scales involved in saline pool formation and mixing are radically different. This difference is not unexpected because flow events are the primary cause of mixing and groundwater intrusion is responsible for formation of the stratification. Secondly, an examination of Figures 9.14, 9.15, 9.16 and 9.17 indicates that the volume of saline water stored below the halocline increases in a linear fashion during low flow periods (at least until the scour depression is full). Therefore mixing is insignificant during these periods. During periods of mixing the rapid reduction in saline water storage indicates that any groundwater inflow is insignificant compared to the mixing. These saline pools can therefore be characterised by formation periods when no significant flushing is occurring and flushing periods when groundwater inflows are insignificant.

This dichotomy between formation and flushing has implications for the application of Salipool. Firstly, it makes simulation of saline pools somewhat easier because the behaviour of the saline pool is dominated by a single process at any one time. This also implies that, on most occasions, the observed formation rate will provide an accurate estimate of the rate at which groundwater contributes to the stratification. Therefore the use of the time-weighted formation rate as an estimate of Q_{gw} is justified.

Secondly, the dichotomy means that the formation and mixing components of the model must be calibrated and tested with data collected at different time-scales. If the flushing coefficient is to be determined by calibration, long term simulations of stratification are unlikely to be of useful. This is due to the fact that, from a seasonal perspective, most significant flow events rapidly flush the saline pool and thus simply act to reset the stratification to zero. The insensitivity of the seasonal behaviour to C_F is demonstrated in Figure 9.18. Calibration of C_F and testing of predicted flushing therefore requires detailed observations of stratification during mixing events. However observations of changes in stratification over lengthy low flow periods are required to characterise saline pool formation.

Given the different time-scales characterising the two processes, data collected from the Wimmera River under the current flow regime are unlikely to be useful for examining interactions between formation and mixing processes that may occur with an altered flow regime. When using Salipool to predict the impact of environmental flows on saline pool formation the dichotomy between formation and flushing periods will disappear. The reason for this is most easily understood by considering an environmental release which maintains a minimum discharge in the river and aims to limit the saline water storage to some small volume. To achieve this a balance between the mixing caused by the river flow and the

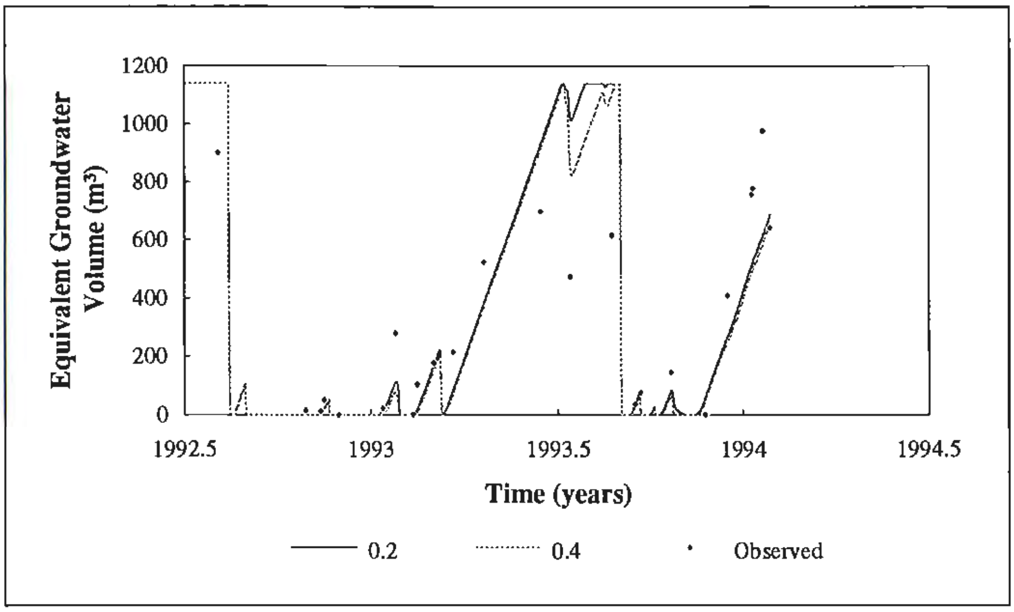


Figure 9.18: Sensitivity of Simulations of stratification at Lower Norton 1 to an increase in the value of C_F from 0.2 to 0.4. Observed stratification is also shown.

contribution of groundwater to the stratification must be achieved. Hence the dichotomy disappears. From a modelling perspective it then becomes imperative to predict both the accumulation and the flushing of saline water correctly.

Furthermore, under circumstances where an intermediate flow limits the accumulation of saline water to a small volume, the assumption that the rate at which saline water enters the saline layer is independent of the amount of stratification may not be justified. It is possible that groundwater accumulates in the scour depression by entering the stream channel and then flowing down the channel bed or banks. If that were so then the groundwater flowing along the channel bed or banks may be mixed by the intermediate flow described above. In that case the rate at which groundwater contributes to the saline layer is likely to increase as the area of bed protected from the main river flow by the density stratification increases.

9.4 SENSITIVITY ANALYSIS OF SALIPOOL.

The sensitivity of Salipool to the flushing parameter and various model inputs was studied so that data and parameter accuracy requirements and simulation errors could be assessed. If it is assumed that the geometry of a given scour depression is well known, the parameters and variables that need to be considered in a sensitivity analysis include g', C_F, U_1, Q_{gw} and the initial conditions.

Some inferences about the sensitivity of Salipool can be drawn from the equations that form the basis of the model. The mixing rate is proportional to C_F, U_1^3, and g'^{-1}. Therefore Salipool is likely to be more sensitive to U_1 and hence the stream discharge than either C_F or g'. The sensitivity to g' and C_F should be similar in magnitude though in the opposite direction. Because changes in V_S depend on the difference between the groundwater inflow rate and the mixing rate, Salipool will be insensitive to Q_{gw} during periods dominated by mixing and during periods of insignificant mixing it will be insensitive to C_F, U_1^3, and g'.

9.4.1 SENSITIVITY DURING FLUSHING.

A series of simulations of flushing were conducted for the saline pools at Lower Norton 1, Lower Norton 2, Polkemmet and Tarranyurk. These simulations began with the scour depression completely full. The stream discharge Q_r was increased from zero at a constant rate of 1 000 Ml/d/d in the standard case and the calibrated model was used for each site. The sensitivity of the simulations to changes of ±20% in dQ_r/dt, C_F and g' was assessed. Figure 9.19 shows the results of the simulations at Lower Norton 1 when dQ_r/dt and C_F were varied. Figure 9.19 shows that the time taken to flush a saline pool is more sensitive to the rate of increase in the stream discharge than to C_F. It should also be noted that the flushing of the saline pool is not particularly sensitive to either of these parameters. Similar results were obtained for other sites.

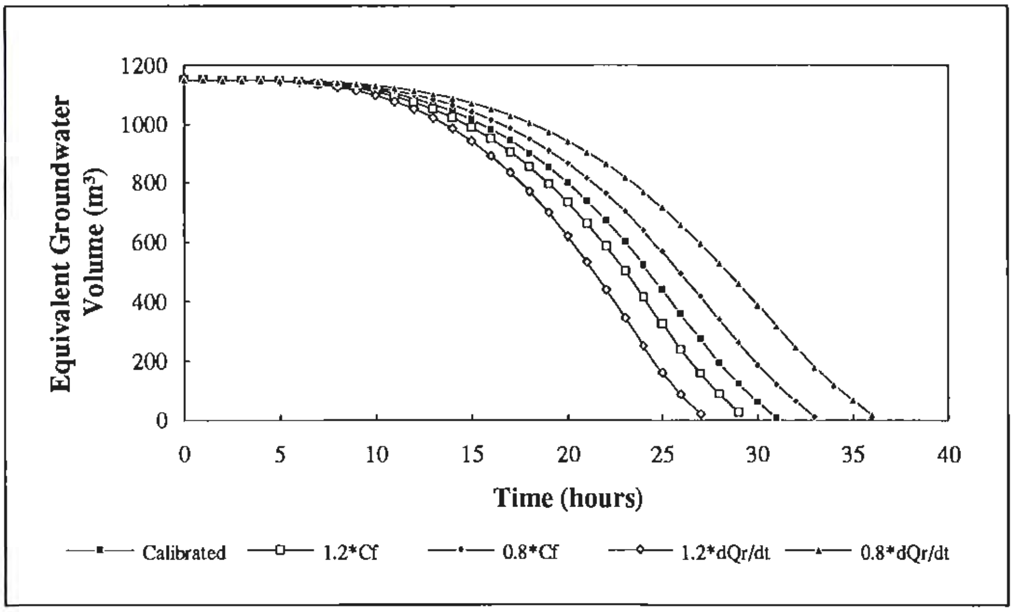


Figure 9.19: Sensitivity of predictions of flushing at Lower Norton 1 to ±20% changes in dQ_r/dt and C_F.

Site	Q_{rf} (Ml/d)	dQ_r/dt		C_F		g'	
		-20%	+20%	-20%	+20%	-20%	+20%
Lower Norton 1	1308	0.34	0.26	-0.32	-0.27	0.32	0.26
Lower Norton 2	2383	0.32	0.28	-0.34	-0.26	0.31	0.28
Polkemmet	1496	0.31	0.26	-0.32	-0.24	0.31	0.26
Tarranyurk	1167	0.33	0.30	-0.36	-0.27	0.32	0.29

Table 9.2: Sensitivity of predictions of saline pool flushing at Lower Norton 1, Lower Norton 2, Polkemmet and Tarranyurk to changes of ±20% in dQ_r/dt, C_F and g' during flushing events.

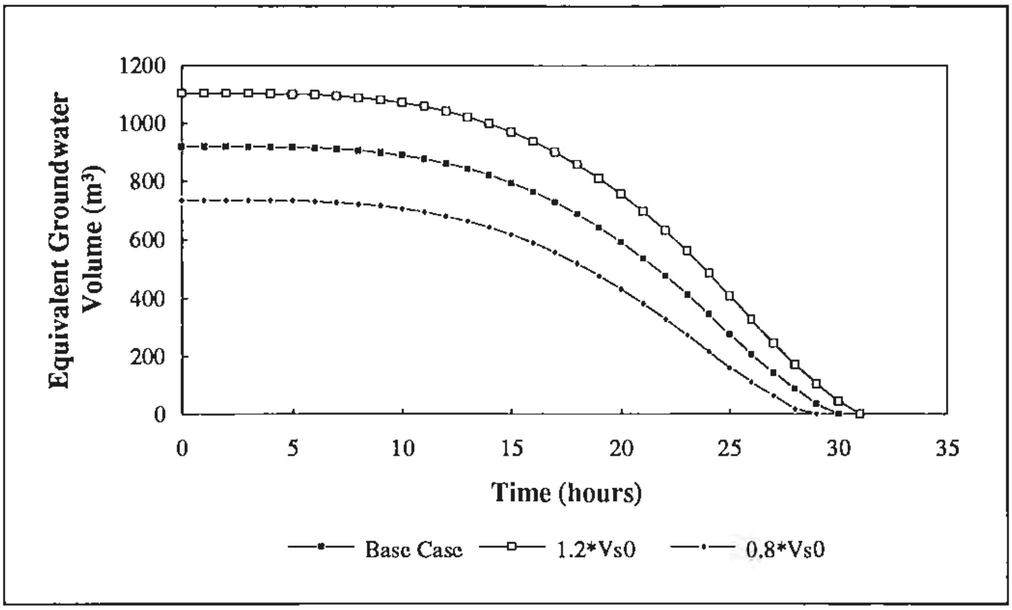


Figure 9.20: Sensitivity of predictions of flushing of Lower Norton 1 to ±20% changes in the initial value of saline water storage, V_{s0}.

Table 9.2 provides sensitivities for all sites and parameters. In Table 9.2 the sensitivity is defined as $(\Delta Q_{rf}/Q_{rf})/(\Delta P/P)$ where Q_{rf} is discharge at the time of complete flushing for the base case, ΔQ_{rf} is the increase Q_{rf} caused by the changed parameter, P is the base parameter value and ΔP is the increase in the parameter. Table 9.2 demonstrates that results were insensitive to parameter changes and were consistent between sites. Results for g' were similar to C_F. The discharge at which flushing is complete is less sensitive to dQ_r/dt than the time taken to flush the pool. This is due to the fact that a more rapid rise in discharge leads to more rapid flushing.

The sensitivity of Salipool to initial conditions was also assessed. An initial saline water volume equal to 80% of the depression volume was used as a base case. This was changed by ±20%. Figure 9.20 shows the results for Lower Norton 1. Results were similar for other sites. Flushing is more insensitive to initial conditions than to dQ_r/dt, C_F and g'.

9.4.2 SENSITIVITY DURING FLOW RECESSION.

During a period of flow recession Salipool predicts the discharge, Q_{ri} , below which stratification begins to develop. The sensitivity of Q_{ri} to changes in C_F, Q_{gw} and g' was assessed for each saline pool. The discharge used in these simulations was specified so that Q_r decreased by 96 Ml/d/d (4 Ml/d/h) and the simulation time-step was 0.25 h. The stream discharge at the time when the saline water storage increased to the small volume of 0.5 m^3 was used as an estimate of Q_i. The calibrated versions of Salipool were used to obtain base values of Q_{ri} and parameters were then varied by ±20%. Table 9.3 provides sensitivities defined as $(\Delta Q_{ri}/Q_{ri})/(\Delta P/P)$ where Q_{ri} refers to the base scenario and ΔQ_{ri} is the increase in Q_{ri} associated with ΔP.

Predictions of Q_{ri} are insensitive to changes in C_F, Q_{gw} and g'; however Q_{ri} is slightly more sensitive than Q_{rf} to changes in C_F and g'. It should also be noted that the magnitudes of the predicted values of Q_{ri} are consistent with stream discharges at each site at times when stratification was first detected (see Tables 8.1, 8.2, 8.3, 8.4, 8.5).

Site	Q_{ri}	Q_{gw}		C_F		g'	
	(Ml/d)	-20%	+20%	-20%	+20%	-20%	+20%
Lower Norton 1	224	0.49	0.49	-0.47	-0.38	0.42	0.38
Lower Norton 2	253	0.45	0.42	-0.47	-0.32	0.42	0.38
Polkemmet	310	0.44	0.39	-0.45	-0.32	0.40	0.35
Tarranyurk	194	0.52	0.41	-0.41	-0.36	0.41	0.34

Table 9.3: Sensitivity of predictions by Salipool of stream discharge at which stratification begins to form at Lower Norton 1, Lower Norton 2, Polkemmet and Tarranyurk to decreases and increases of 20% in Q_{gw}, C_F and g' during flushing events.

9.4.3 SENSITIVITY DURING STEADY STREAM DISCHARGE.

Since one possible application of Salipool is to the assessment of environmental flows the sensitivity of Salipool's predictions during periods of steady low flow was assessed. The steady stream discharge was set to $0.5*Q_{ri}$ for each site, the site was assumed to be unstratified initially and calibrated values for other parameters were used. The sensitivity of Salipool to changes of ±20% in Q_r, Q_{gw}, C_F and g' was assessed.

Figures 9.21, 9.22, 9.23 and 9.24 show the simulations for each site when Q_r and Q_{gw} were varied. Because saline pools develop during periods of low flow the saline water storage after a typical low flow season was used to characterise the stratification. A time of 250 days was chosen since low flow conditions generally exist throughout the period from November to June although this is highly variable. The saline water storage on the 250th day of simulation is V_{s250}.Table 9.4 provides sensitivities defined as $(\Delta V_{s250}/V_{s250})/(\Delta P/P)$ where V_{s250} refers to the base scenario and ΔV_{s250} is the increase in V_{s250} associated with ΔP. Sensitivities were not calculated if the scour depression filled before 250 days had elapsed.

The predicted development of the saline pools during a steady low flow is sensitive to parameter changes in that the proportional change in V_{s250} was often greater than the proportional change in the parameter. V_{s250} was significantly more sensitive to parameter changes than either Q_{ri} or Q_{rf}. Predictions were most sensitive to Q_r, followed by Q_{gw} and then C_F and g'. It should be noted that sensitivities varied between sites and that the time taken for individual scour depressions to fill or for the saline water storage to reach a steady state also varies significantly between sites; generally being larger for larger scour depressions and smaller groundwater discharges.

Site	V_{s250} (m^3)	Q_r		Q_{gw}		C_F		g'	
		-20%	+20 %	-20%	+20 %	-20%	+20 %	-20%	+20 %
Lower Norton 1	833	na	-2.95	1.76	na	-1.47	-1.24	1.50	1.21
Lower Norton 2	3311	-0.84	-1.02	1.17	1.17	-0.35	-0.33	0.41	0.29
Polkemmet	530	na	na	na	na	na	na	na	na
Tarranyurk	526	-3.75	-3.20	1.63	1.63	-1.39	-1.12	1.37	1.13

Table 9.4: Sensitivity of predictions by Salipool of saline water storage volume after 250 days of steady stream discharge equal to 0.5*Qi at Lower Norton 1, Lower Norton 2, Polkemmet and Tarranyurk to changes of ±20% in Q_r, Q_{gw}, C_F and g'. na - Not available due to scour depression filling before 250 days had elapsed.

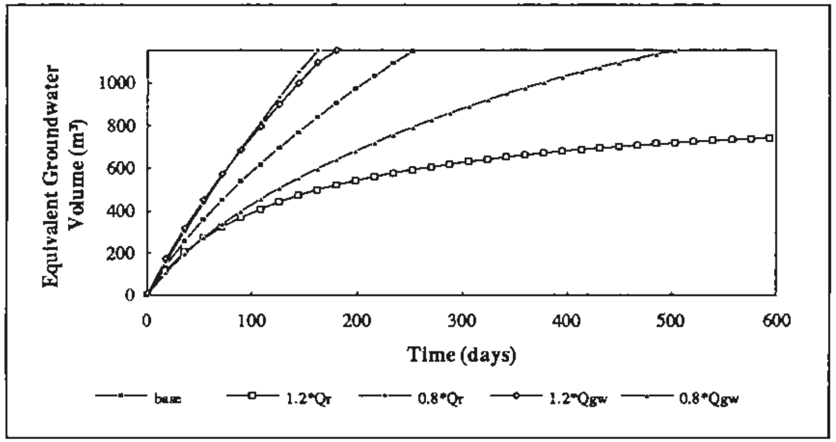


Figure 9.21: Sensitivity of predictions of saline pool development at Lower Norton 1 to ±20% changes in Q_r and Q_{gw}.

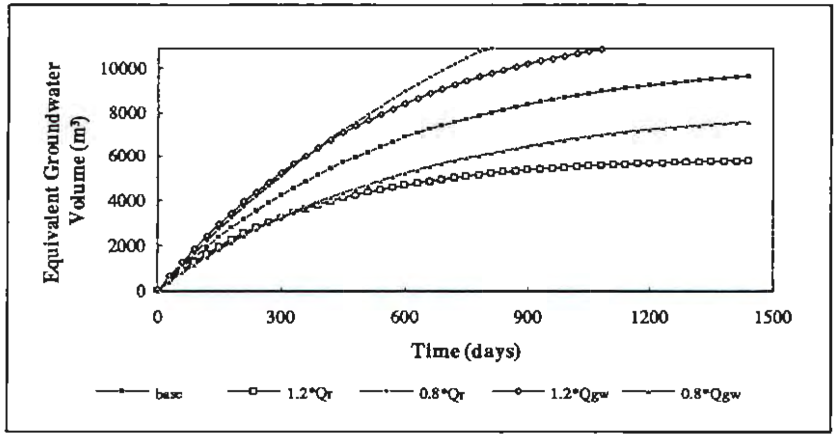


Figure 9.22: Sensitivity of predictions of saline pool development at Lower Norton 2 to ±20% changes in Q_r and Q_{gw}.

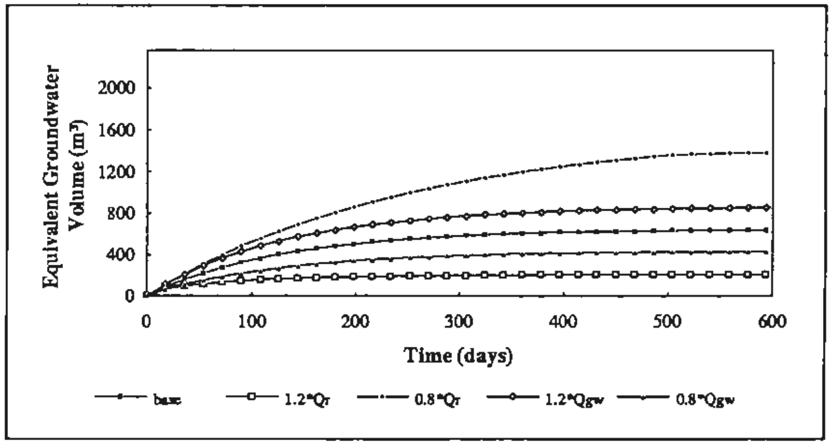


Figure 9.23: Sensitivity of predictions of saline pool development at Tarranyurk to ±20% changes in Q_r and Q_{gw}.

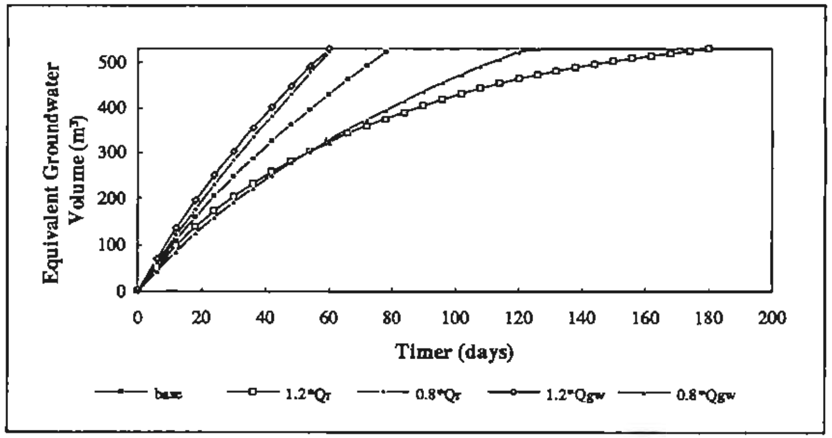


Figure 9.24: Sensitivity of predictions of saline pool development at Polkemmet to ±20% changes in Q_r and Q_{gw}.

9.4.4 IMPLICATIONS OF OBSERVED SENSITIVITIES.

The above sensitivity analysis shows that predictions of the flushing of saline pools and the flow below which stratification develops are insensitive to changes in the mixing coefficient or the buoyancy. However, predictions of the development of stratification under a steady low flow can be sensitive to these parameters. The sensitivity analysis indicates that the rate at which the discharge increases is the most important determinant of the time required to flush a saline pool.

The greater sensitivity during the steady flow simulations is due the importance of both the groundwater inflow rate and the rate of mixing in determining V_S combined with variations in the mixing rate associated with changes in V_S. The variation in V_S was also less constrained during these simulations. When the flushing of saline pools was simulated the groundwater inflow rate was set to zero therefore did not affect the simulation. These simulations began with the depression full and finished when the depression was completely flushed. Variations in V_S were unimportant when determining Q_{ri} because the depression was empty or almost empty during the entire simulation.

Salipool was most sensitive to parameter and variable changes when simulating the development of stratification during steady low flows. In this case predictions were more sensitive to stream discharge and groundwater discharge than to C_F and g'. This implies that the accuracy of groundwater discharges will be more important than the accuracy of C_F and g' in assessing the impact of environmental flows. It also implies that small increases in an environmental flow can have a significant impact on stratification in some circumstances. For example if a steady flow of 100 Ml/d meant that V_S reached an equilibrium such that the scour depression was half full of saline water then increasing that flow to 120 Ml/d might halve the equilibrium storage.

9.5 ENVIRONMENTAL FLOW ASSESSMENT.

Salipool was used to simulate the effect of a simple environmental release strategy on the saline pools at Lower Norton 1, Lower Norton 2, Polkemmet and Tarranyurk. These simulations were primarily conducted to demonstrate a possible application of the model and are not intended as a basis for a detailed study of environmental flow release policies. For these simulations the discharge measured at Horsham was assumed to be representative of the river between Horsham and Lake Hindmarsh. A simple environmental release policy was

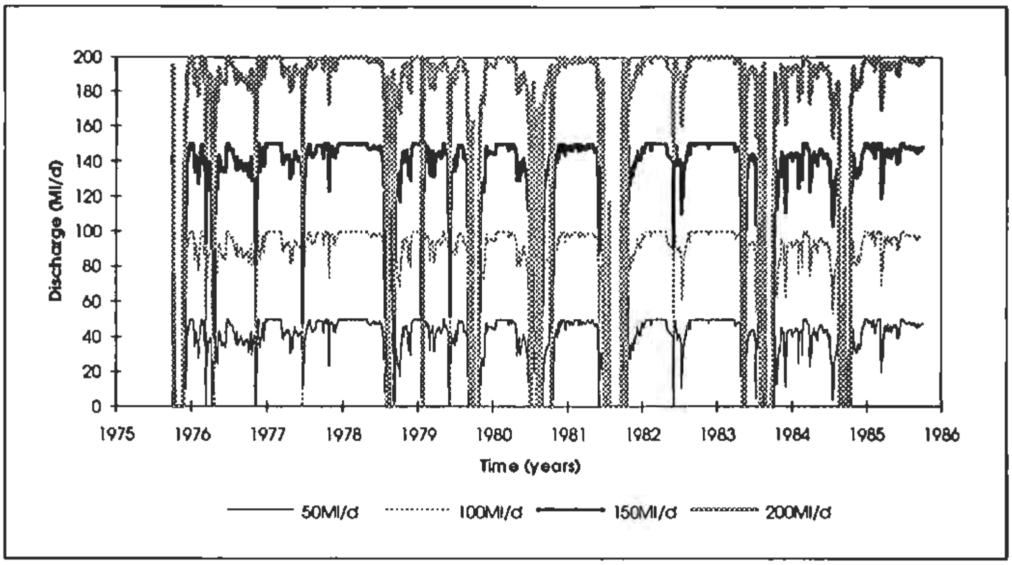


Figure 9.25: Environmental realease required to maintain minimum stream discharges of 50 Ml/d, 100 Ml/d, 150 Ml/d and 200 Ml/d at Horsham during the period 1/10/75 to 30/9/85.

incorporated into the discharge at Horsham by assuming flow releases were made to prevent the stream discharge falling below a minimum value. Simulations for the period 1/10/75 to 30/9/85 were run with the monitored flows and with flow minima of 50 Ml/d, 100 Ml/d, 150 Ml/d and 200 Ml/d imposed. Figure 9.25 shows the hypothetical flow releases at Horsham. Average annual flow releases required during the simulation period were 13 700 Ml, 30 100 Ml, 46 700 Ml and 63 600 Ml for the 50 Ml/d, 100Ml/d, 150 Ml/d and 200 Ml/d minima respectively. It is noted that these releases are in excess of currently uncommitted supply and that the largest release represents 40% of the total deliveries of the WMSDS.

Figure 9.26 shows the results of these simulations. The seasonal pattern of stratification and the distinction between formation and mixing periods can be clearly seen in the simulations using the monitored flows. It is noted that during extensive low flow periods such as between 1975 and 1978 the scour depressions at all sites fill completely. The stratification would extend between scour depressions on these occasions and stratification due to saline water overflowing from other saline pools is likely to affect a number of scour depressions that do not normally become stratified. The current model is not capable of accurately simulating interactions between scour depressions.

The simulations show that the effect of a specific environmental flow policy varies between sites. At Tarranyurk a minimum flow of 100 Ml/d limits the development of stratification significantly while at Polkemmet an environmental flow of 200 Ml/d only just prevents the scour depression being entirely filled. It should be noted that the impact of flows less than 200 Ml/d cannot be determined at Polkemmet using the present model because the scour depression fills completely. Significant benefits may still be obtained at this site by limiting the development of more extensive stratification. The simulations also show that significant benefits can be obtained by small increases in flow under certain conditions. For example at Lower Norton 1 a minimum discharge of 100 Ml/d only slows the development of stratification slightly while 150 Ml/d limits V_S to less than 20% of the total volume of the scour depression.

It should be noted that predictions made by Salipool with a flow regime similar to that used in these environmental flow simulations have not been tested. While similar simulations may provide useful information for developing an environmental flow management strategy, the impact of releases made under such a strategy should be monitored and the strategy would need to be modified as further information was obtained.

9.6 MIKE 11-SP.

The Advection-Dispersion component of MIKE 11 has been modified to incorporate a description of saline pools at specific locations along a stream. For one-dimensional Advection-Dispersion simulations each computational grid point in MIKE 11 represents a finite volume of water. Solute is transported into and out of this finite volume by advection and dispersion and by lateral inflows and outflows (DHI, 1992b). To incorporate a description of saline pools this finite volume was divided into two parts: a fresh upper compartment and a saline lower compartment (Figure 9.27). Groundwater flows into the river reach represented by the finite volume at a rate Q'_{gw}. A proportion, φ, of the groundwater is assigned to the saline layer and the remainder is assigned directly to the freshwater layer. Saline water is transferred from the saline to the fresh layer at the rate Q_{mix}. This model is referred to as MIKE 11-SP. MIKE 11-SP consists of a one-dimensional Advection-Dispersion model coupled to a series of local saline water stores which simulate saline pools.

9.6.1 SALINE LAYER DESCRIPTION.

In MIKE 11-SP the saline layer is described using the algorithms incorporated in Salipool with some minor modifications. These algorithms were described in §9.2 and only the differences, which are related to the calculation of the mixing

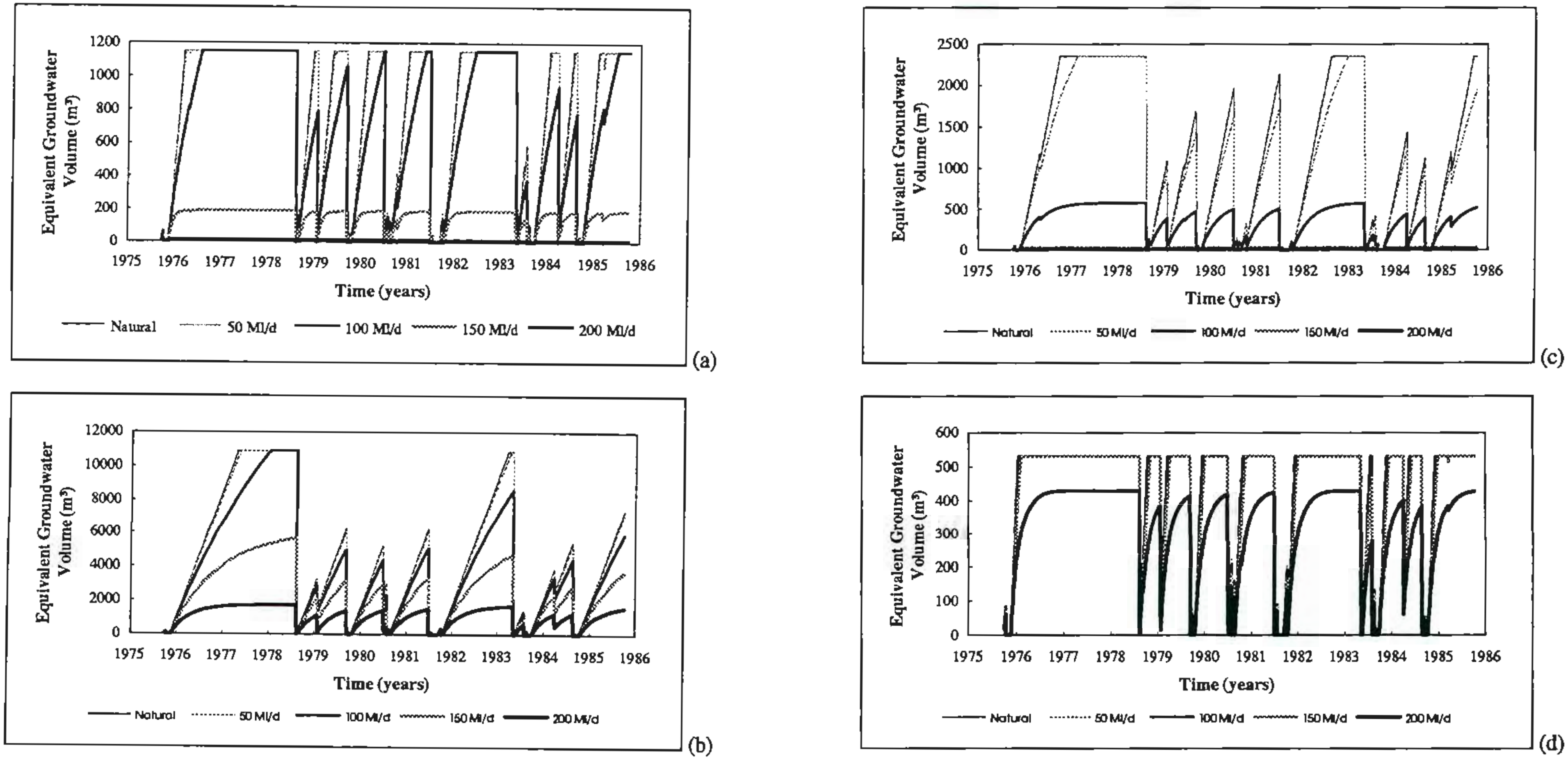


Figure 9.26: The effect of maintaining minimum stream discharges of 50 Ml/d, 100 Ml/d, 150 Ml/d and 200 Ml/d during the period 1/10/75 to 30/9/85.at (a) Lower Norton 1 (b) Lower Norton 2, (c) Tarranyurk and (d) Polkemmet.

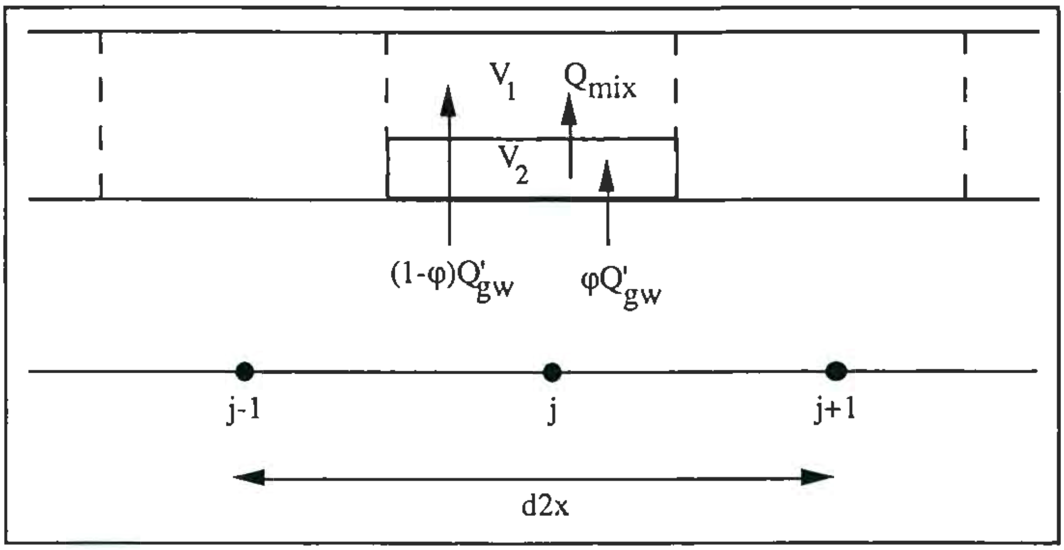


Figure 9.27: Inclusion of saline pools in MIKE 11-SP. A saline pool at computational point j is represented using a fresh layer of volume V_1 and a saline layer of volume V_2. Groundwater contributes to saline and fresh layers at the rate $\varphi Q'_{gw}$ and $(1-\varphi)Q'_{gw}$ respectively and mixing occurs at the rate Q_{mix}.

rate and the description of the disappearance of the lower layer and periods of extreme stratification, are discussed here. In MIKE 11-SP the mixing rate is calculated for time n+1/2 rather than at times n and n+1. The mean upper layer velocity is calculated as

$$U_1^{n+1/2} = \left| Q^{n+1/2} \frac{0.5*d2x}{V_1^{n+1/2}} \right| \tag{9.8}$$

where $Q^{n+1/2}$ is the stream discharge time n+1/2, d2x is the distance between the grid points either side of the point where the saline pool is located and $V_1^{n+1/2}$ is the volume of the fresh layer at time n+1/2. $V_1^{n+1/2}$ is calculated as $V_{tot}^{n+1/2} - 0.5*(V_2^{n+1}+V_2^{n})$ where V_{tot} is the combined volume of the fresh and saline layers (determined from hydrodynamic simulation) and V_2 is the volume of the saline layer. The Richardson number is calculated using

$$Ri_L^{n+1/2} = \frac{g'\,0.5(L^n+L^{n+1})}{U_1^{n+1/2}} \tag{9.9}$$

and Equation 9.2 is used to calculate $Q_{mix}^{n+1/2}$ with $A_p = 0.5*(A_p^{n+1} + A_p^{n})$. An iteration is used to obtain estimates of the geometric variables (L, V, A) at time n+1.

During extended low flow periods the volume saline water layer at some saline pools can increase so that it fills the entire channel at that location. Since this causes difficulties with the simulation of the upper layer (because its volume becomes zero or negative), Q_{mix} was set equal to $\varphi Q'_{gw}$ when V_2 exceeded $0.9*V_{tot}$. This prevents V_2 increasing any further but retains a saline layer which almost fills the entire channel. Thus when a fresh flow moves down the river it initially spreads across the top of the saline layer then starts mixing the saline layer. This is qualitatively consistent with observed behaviour (see Chapter 8.9.1).

The saline layer in both Salipool and MIKE 11-SP can disappear during high flow periods. When the volume of the saline layer in Salipool becomes negative its volume is simply set to zero. However in MIKE 11-SP the mixing rate is modified to

$$Q_{mix}^{n+1/2} = \varphi Q'_{gw} + V_2^n/\Delta t \, . \qquad (9.10)$$

This ensures that the total mass of solute is conserved.

9.6.2 FRESH LAYER DESCRIPTION.

The upper or fresh layer in MIKE 11-SP is simulated using the Advection-Dispersion equation and similar numerical algorithms to those incorporated in MIKE 11. These algorithms are described in detail in Chapter 3.6.3 and Appendix 1. Two differences exist: the calculation of the volume and the specification of the lateral inflow. In MIKE 11-SP the volume of the upper layer is calculated at any point in time as $V_1 = V_{tot} - V_2$ and V_1 is used to specify the volume used in the Advection-Dispersion equation (see Appendix 1).

The discharge for the lateral inflow into the upper layer is calculated as a combination of the rate at which groundwater enters the upper layer and the mixing rate from the lower layer. Thus the lateral inflow is

$$q = (1 - \varphi)Q'_{gw} + Q_{mix} \, . \qquad (9.11)$$

The concentration of the lateral inflow is constant and equal to the groundwater salinity at the location of the saline pool.

9.7 APPLICATION OF MIKE 11-SP TO THE WIMMERA RIVER.

MIKE 11-SP has been applied to the reach of the Wimmera River between Horsham and Lochiel. Saline pools occur frequently along this section the river but only in one small area upstream of Horsham. Since detailed geometrical data were available only for a small number of scour depressions an idealised scour depression was created for this application. Two sets of simulations were then performed. The first used groundwater discharges obtained from D. Strudwick (DCE, Per. Com.) (see §4.9) which were applied to all reaches of the river. The second used groundwater discharges estimated on the basis of observed saline pool formation rates and only included groundwater inflows at locations where saline pools were included in the river description.

9.7.1 THE IDEALISED SCOUR DEPRESSION.

Figure 9.28 shows the variation of the plan area with elevation above the channel invert for scour depressions at Lower Norton 1, Lower Norton 2, Polkemmet, Big Bend and Tarranyurk. The bend in the river channel at Big Bend is unusually sharp and it is considered that the scour depression there is not representative of most scour depressions between Horsham and Lochiel. Therefore the scour depression geometry at Big Bend was not considered when developing the typical scour depression geometry. It was assumed that only one saline pool existed in any reach described by a single computational grid point and that no interaction occurs between saline pools at neighbouring grid points. It is likely that both these assumptions are violated in specific cases, especially where large groundwater inflows occur; however it is believed that the model captures the main effects of saline pools on the solute transport down the river.

For elevations less than 4 m the plan areas at the remaining four locations were used as a guide to estimate the typical plan area. At an elevation of 7 m it was assumed that the saline pool would exist along the entire reach of river described by the grid point (typically 500 m) and that the width to depth ratio would be 8 (see Chapter 5), thus the plan area is 28 000 m^2. Plan areas at elevations of 5 m and 6 m were then selected to obtain a smooth transition between those at 4 m and 7 m. To obtain the interface length it was assumed that the ratio of the interface width (L/B_i) to the interface length was 8 which is typical of the scour depressions used to estimate the plan areas (Figure 9.29).

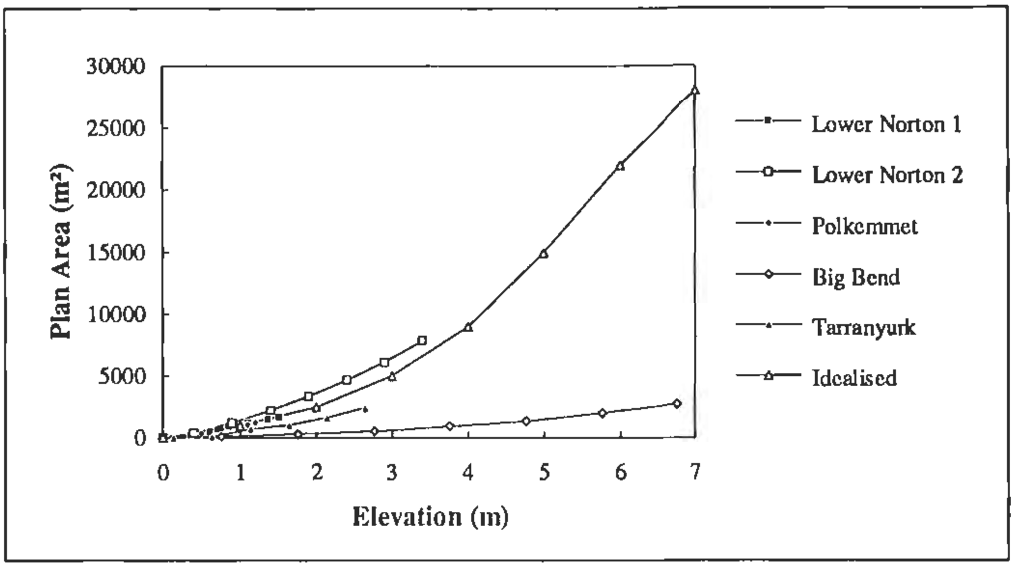


Figure 9.28: The relationship between Plan Area and Elevation for scour depressions at Lower Norton 1, Lower Norton 2, Polkemmet, Big Bend and Tarranyurk. The relationship used for the idealised scour depression is also shown.

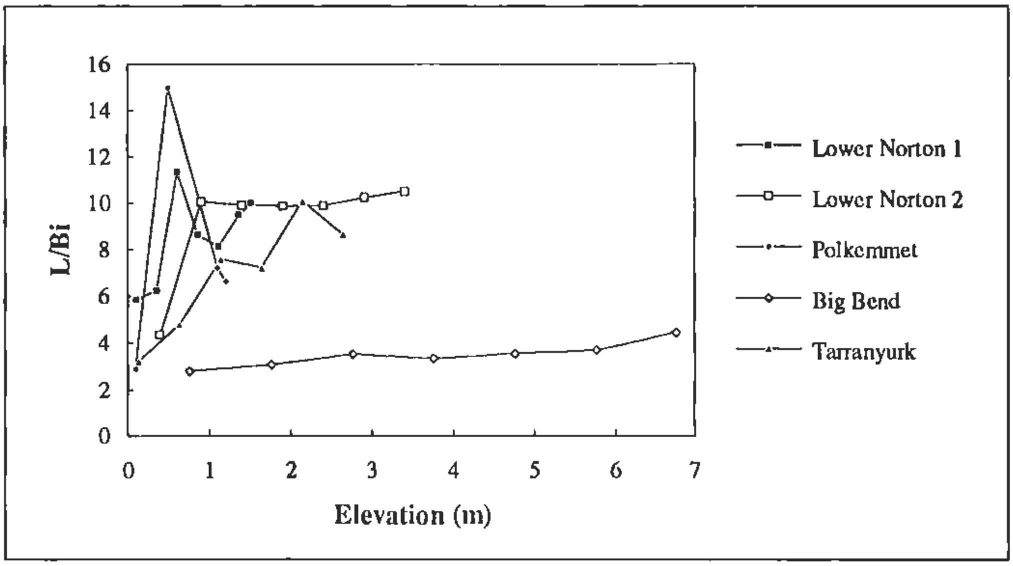


Figure 9.29: Relationship between scour depression length to width ration for scour depressions at Lower Norton 1, Lower Norton 2, Polkemmet, Big Bend and Tarranyurk.

9.7.2 LOCATION OF SALINE POOLS.

It was assumed that saline pools existed at all locations where there was a significant depth of water (>~1.5 m) during low flow periods and where significant groundwater discharges were indicated by the data supplied by Strudwick (DCE, per. com.). Figure 9.30 shows the stream channel invert and the location of saline pools used in the model. A value of $C_F = 0.4$ was used to

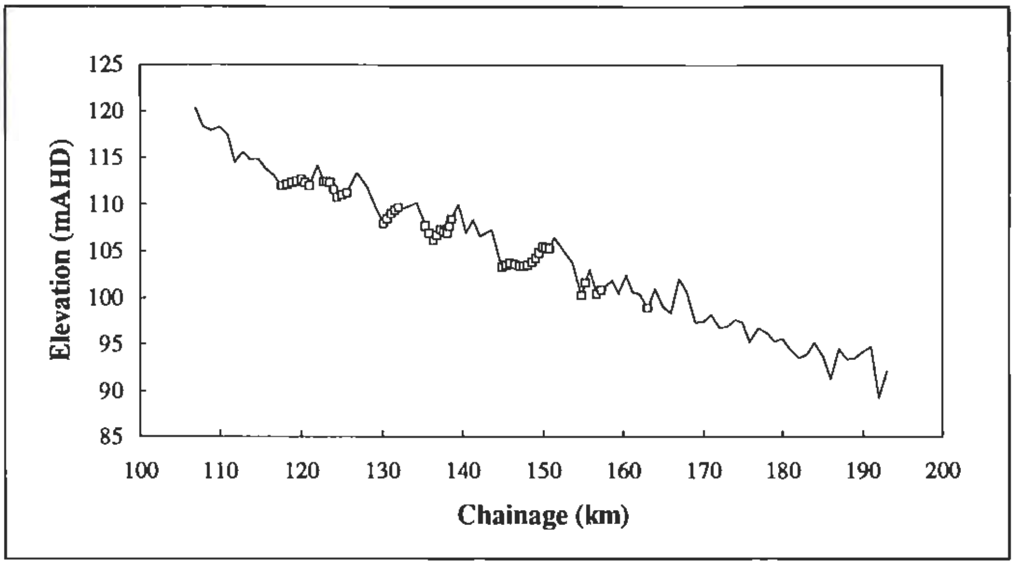


Figure 9.30: Long-section of the Wimmera River between Horsham and Lochiel. The locations of saline pools specified in MIKE 11-SP are shown by the squares.

specify the mixing relationship for each saline pool. This is typical of values observed at saline pools that have been studied in detail in the Wimmera River (see Table9.1).

9.7.3 SIMULATIONS USING MIKE 11-SP.

A series of simulations including saline pools were conducted for the Wimmera River between Horsham (415200) and Lochiel (415246) using MIKE 11-SP. The MIKE 11-SP model was similar to the equivalent reach in the MIKE 11 model of the Wimmera River between Glynwylln and Lochiel. The later model is referred to as the one-dimensional model of the Wimmera River. The main differences between the one-dimensional model and the MIKE 11-SP model relate to the inclusion of saline pools and slight differences in the specification of groundwater inflows. Simulations using the one-dimensional model (Chapter 6) were used to specify the flow and salinity in the Wimmera River at Horsham for the MIKE 11-SP model.

Groundwater inflows were specified in the MIKE 11-SP model in two different ways. At locations without saline pools groundwater influxes were specified as lateral inflows which were identical to those specified in the one-dimensional model of the Wimmera River. Where saline pools were included, the groundwater inflow was specified to each grid point using the subroutine describing saline pools. This means that groundwater inflows were specified at different grid points to those used in the one-dimensional model. In the one-

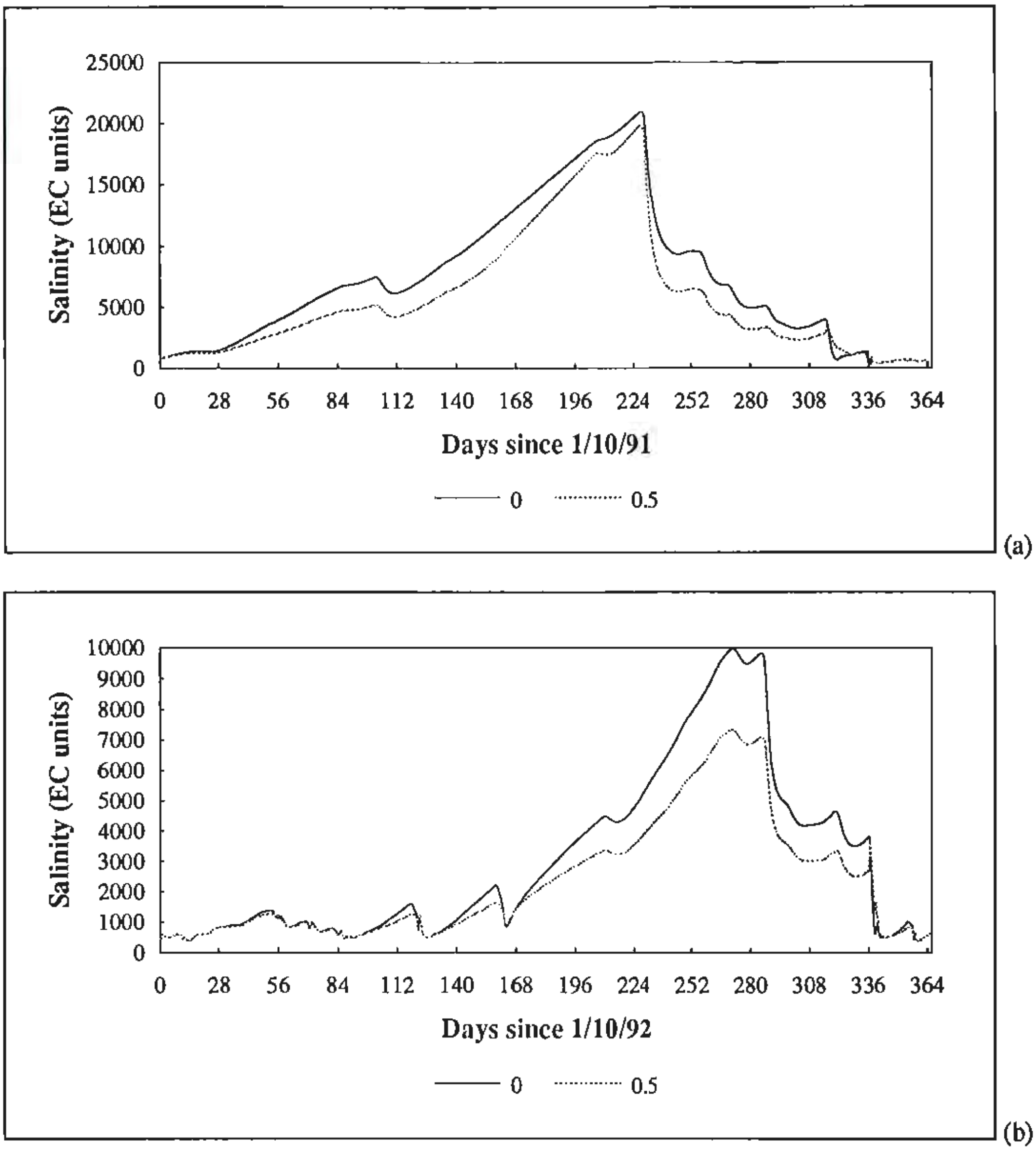
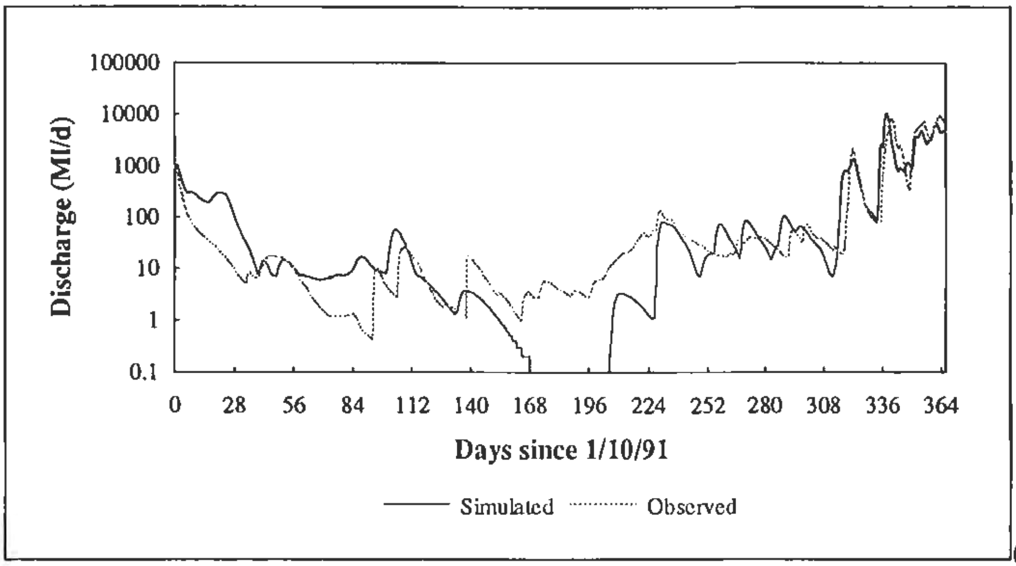
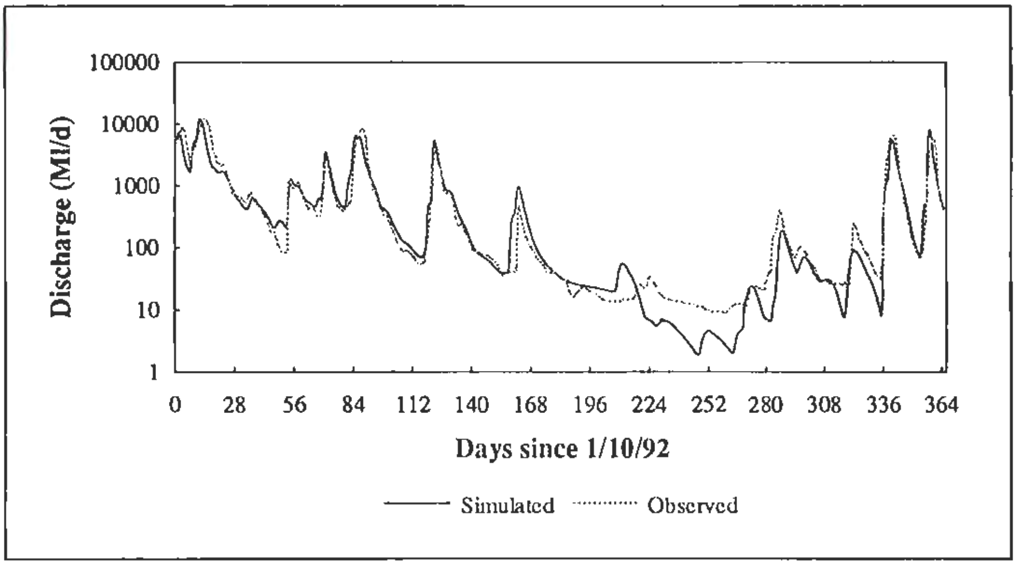


Figure 9.31: Simulated surface layer salinities for the Wimmera River at 159.95 km with φ=0.0 and φ=0.5 for the periods (a) 1/10/91 to 30/9/92 and (b) 1/10/92 to 30/9/93. Groundwater influxes specified using Strudwick's estimates.

dimensional model groundwater inflows were specified at locations which were H-points in the hydrodynamic simulations whereas in the MIKE 11-SP model saline pools and thus groundwater inflows were specified at both H-points and Q-points. The rate of groundwater inflow to each grid point representing a saline pools was specified using the groundwater inflows per unit length in Table 4.6 multiplied by the channel length represented by the particular grid point. Groundwater salinities were obtained from Table 4.6. Small differences in the



(a)



(b)

Figure 9.32: Simulated and observed discharge for the Wimmera River at Upstream of Dimboola for the periods (a) 1/10/91 to 30/9/92 and (b) 1/10/92 to 30/9/93.

total groundwater inflow exist between the one-dimensional model and the MIKE 11-SP model.

Simulations using the MIKE 11-SP model were conducted with different values of φ for the period 1/10/91 to 30/9/93. High river flows existed at the start and end of this period and the stream was unstratified at these times. The simulations include two extended periods of low flow and mixing events following each of these. Figure 9.31 shows the results of the simulations for $\varphi = 0.0$ and $\varphi = 0.5$ for the Wimmera River at chainage 159.95 km and Figure 9.32 shows the variation in simulated discharge during the simulation period. Forty-five of the forty-six saline pools included in the model are upstream of this point which is itself a short distance upstream of the gauging station Upstream of Dimboola (415256). This location is used in preference to the location representing the gauging station since it is more representative of the salinity of water in the large pools in this part of the river. The pattern of temporal variation in the salinity of the surface water is similar along the reach of river between Horsham and Upstream of Dimboola; however during low flow periods surface salinities increase downstream because of the significant influence of groundwater inflows. During high flow periods the stream salinity is dominated by the upstream inflow.

For the first simulation all the groundwater enters the fresh water layer. Therefore the first simulation assumes that the salt is completely mixed across the cross-section and is essentially a one-dimensional simulation. For the second simulation 43% of the total salt flux entering the Wimmera River between Horsham and Lochiel is assigned to the saline layer in a saline pool. A value of $\varphi=0.5$ was chosen arbitrarily to represent saline pools since it is known that some but not all of the groundwater entering the Wimmera River contributes to saline pools. Simulations with other values of $\varphi=0$ are qualitatively similar to those for $\varphi=0.5$.

Comparing the simulations it can be seen that including saline pools in the model ($\varphi = 0.5$) decreases the surface water salinity during low flow periods. This is expected since saline pools act to store salt. When a mixing event occurs the temporary increases in the surface salinity occur. These are not evident in simulations for which $\varphi = 0.0$. Figure 9.33 shows details of the variation in surface salinity for chainages of 143.6 km and 148.1 km and the stream discharge at a chainage of 143.6 km for one of these mixing events. These two locations are upstream and downstream of one of the two reaches with the largest groundwater inflow. Figure 9.34 shows the volume of water stored in typical saline pools for the same period. Figures 9.33 and 9.34 indicate that mixing of saline pools during the rising limb of the hydrograph produces these temporary increases in salinity.

It is noted that the above simulations and those conducted in Chapter 6 predict salinities during low flow periods that are significantly higher than those observed at Upstream of Dimboola. Observations made at other locations during field visits (for example the longitudinal surveys described in Chapter 8.6) also suggest that the river does not become as saline as the simulations predict during

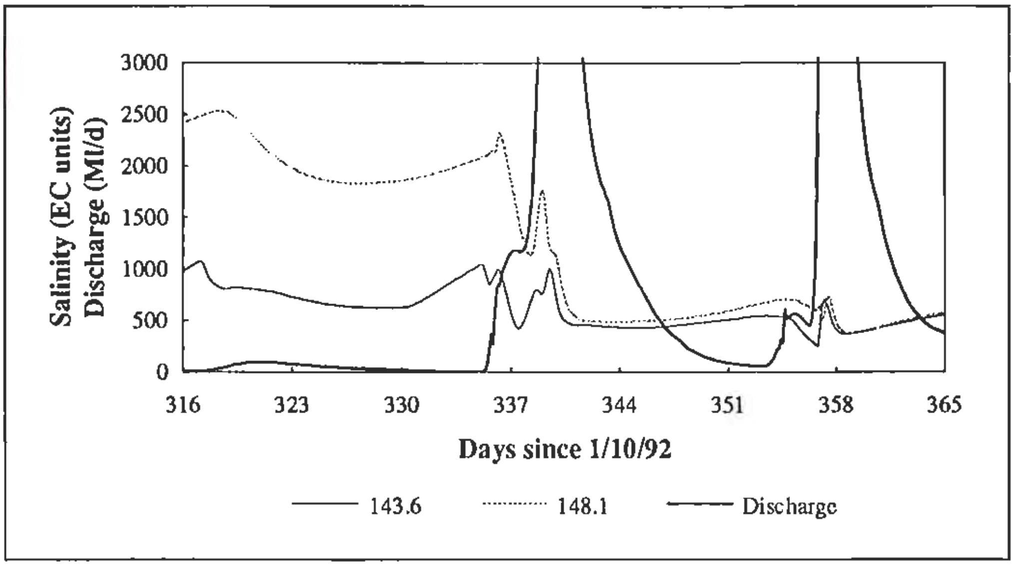


Figure 9.33: Variation in salinity of the surface layer for the Wimmera River at 143.6 km and 148.1 km during a mixing event. Simulated discharge at 148.1km is also shown.

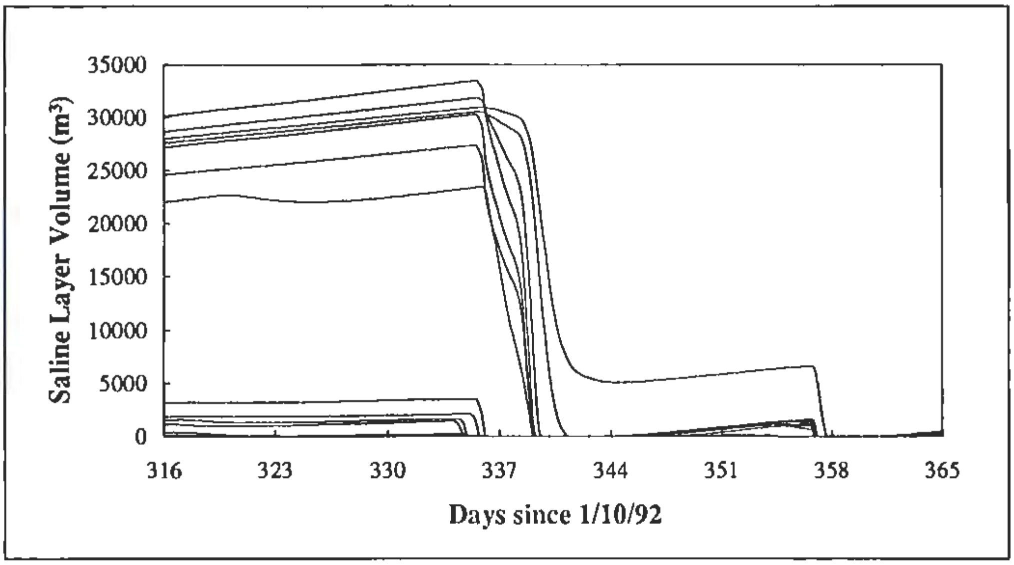


Figure 9.34: Simulated volumes for the saline layer of typical saline pools specified in MIK 11-SP during a mixing event.

low flow periods and that the volume of saline water in saline pools is less than the simulations predict. This may be due to errors in the estimates of groundwater inflow rates provided by Strudwick (DCE, per. com.). Further simulations were conducted with new estimates of the groundwater inflow rates so that the impact of errors in the groundwater flow rates could be examined and so that the behaviour of the stream under different groundwater inflows could be examined.

Site	Scour Depression Length (m)	Q_{gw} (Ml/d)	q_{gw} (Ml/d/km)
Lower Norton 1	130	0.010	28
Lower Norton 2	287	0.021	27
Polkemmet	90	0.011	45
Tarranyurk	145	0.008	20

Table 9.5: Estimates of groundwater inflow per unit length for four sites on the Wimmera River downstream of Horsham.

9.7.4 SIMULATIONS WITH GROUNDWATER INFLOWS ONLY AT SALINE POOLS.

Groundwater inflow rates to four saline pools were estimated in Chapter 8.9.2.2. By dividing these by the length of the scour depression estimates of the groundwater inflow per unit length q_{gw} were obtained (Table 9.5). Two assumptions must be valid for these estimates to be representative of groundwater inflows to the river. Firstly all the groundwater entering the stream along the length of the scour depression must accumulate below the halocline and secondly the rate at which groundwater enters the river at the scour depression must be representative of that reach. The validity of the first assumption is unclear. If it were incorrect the groundwater inflow rate would be underestimated.

It is probable that the second assumptions is incorrect. Saline pools do not form in all scour depressions. However the locations where groundwater inflow rates have been estimated were selected because saline pools do form there. Therefore, assuming that saline pools form where there is a significant groundwater inflow, it would be expected that groundwater inflows estimated at saline pools would overestimate the average groundwater inflow per unit length. However it should be noted that saline pools do not necessarily form in scour depressions where prominent bank seepage exists (see Chapter 8).

There are likely to be errors in the groundwater inflow rates estimated on the basis of observed saline pool behaviour. Nevertheless the mean of these estimates (30 Ml/d/km) has been used as an alternative estimate of the groundwater inflow to the Wimmera River between Horsham and Lochiel. This groundwater inflow rate was applied at all saline pools in the MIKE 11-SP model. Groundwater inflows were set to zero elsewhere. The salinity of the groundwater was assumed to be 10 000 EC for chainages between 106.9 km (Horsham) and 145.0 km and 35 000 EC downstream of 145 km. In these

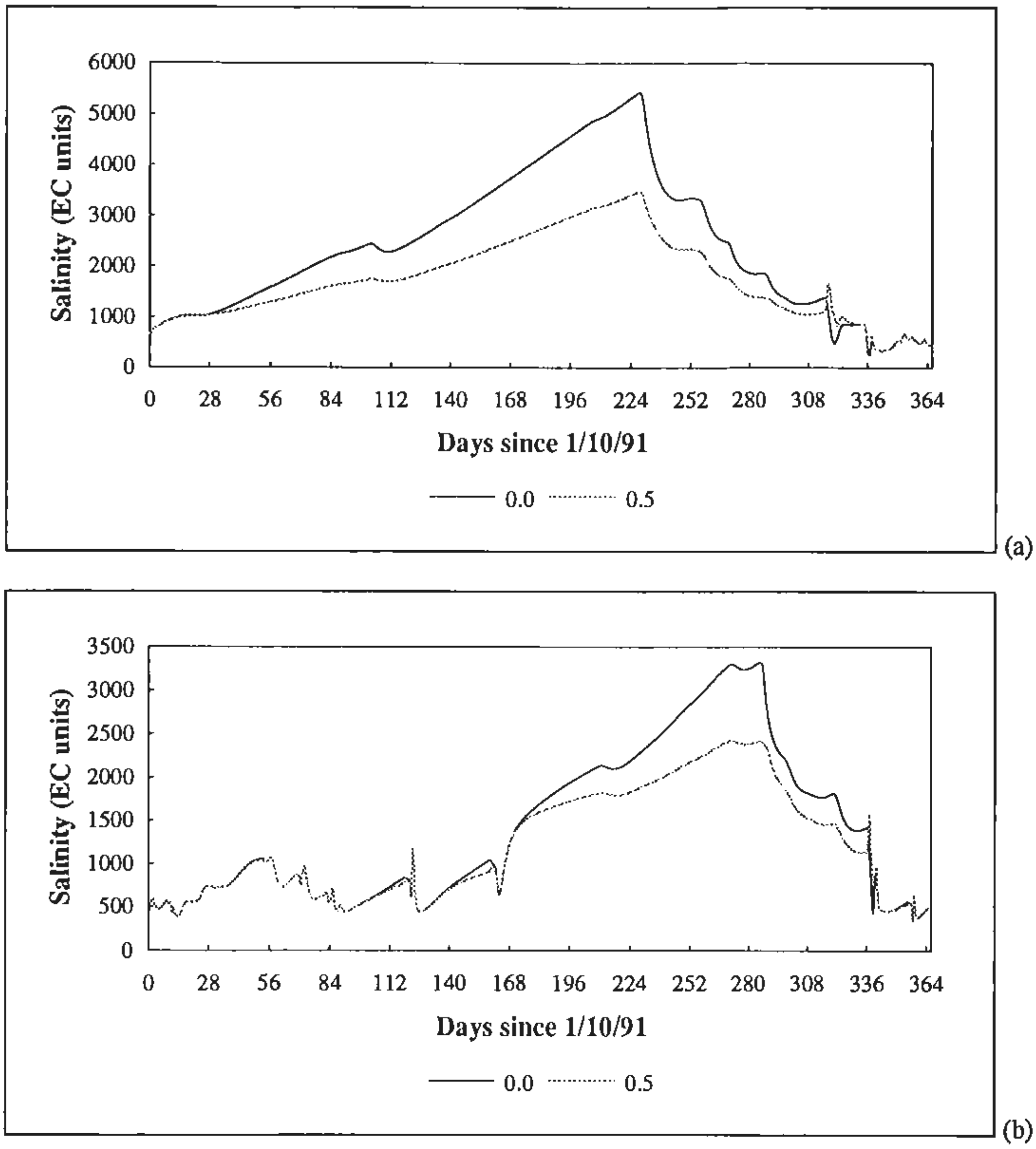


Figure 9.35: Simulated surface layer salinities for the Wimmera River at 159.95 km with φ=0.0 and φ=0.5 for the periods (a) 1/10/91 to 30/9/92 and (b) 1/10/92 to 30/9/93. Groundwater influxes specified using estimates based on saline pool formation rates.

simulations the total salt flux entering the model due to groundwater inflows was 21 t/d compared to 75 t/d for the simulations described in §9.7.2.

Figure 9.35 compares simulations using the new groundwater inflow rates and φ = 0.0 and φ = 0.5. The temporal pattern of salinity variation for these simulations is similar to that for those using Strudwick's (DCE, Per. Com.) groundwater inflow rates; however the surface salinities during low flow periods are approximately 4 times smaller, the difference between φ = 0.0 and φ = 0.5 is larger and the increase in salinity during mixing events is larger. These differences are directly related to the smaller groundwater inflow rates. A comparison of the two sets of simulations indicates that accurate estimates of groundwater influxes are essential for accurate simulations of stream salinity during low flow periods in the Wimmera River downstream of Horsham.

Compared with the first set (§9.7.2), the second set of simulations (§9.7.3) produce lower and thus more realistic stream salinities during low flow periods. This suggests that the total salt influx due to groundwater estimated on the basis of the observed behaviour of the four saline pools may be more accurate. However errors in the salinity simulations arising from dilution effects also occur due to errors in the simulation of discharge and errors in the volume of water stored in the stream channel during low flow periods. Therefore it is possible that significant errors in the total groundwater inflow exist in both cases. Furthermore the simulations do not give an indication of the accuracy of the spatial distribution of groundwater inflows.

9.7.5 OBSERVED SALINITY BEHAVIOUR DURING MIXING.

Both of the simulations with $\varphi = 0.5$ (§9.7.3 and §9.7.4) (ie. those including saline pools) indicate that pulses of salt occur on the rising limb of some hydrographs. This temporary increase in salinity, which is associated with mixing of saline pools, requires a low flow period so that stratification can develop followed by a hydrograph with a peak flow that is sufficiently large to cause significant mixing. It is most prominent after lengthy low flow periods.

The continuous salinity record for the Wimmera River at Upstream of Dimboola (415256) was examined for salinity pulses associated with flow events; however no such pulses were found. Unfortunately the salinity record has a large amount of missing data so this comparison of the model with observed salinity variation during mixing events is not conclusive. However in 1991 only one mixing event occurred. This event followed a low flow period that extended for 9 months and had a peak discharge of 2 800 Ml/d. The predicted salinity pulses are most significant for this type of event (ie following a long low flow period and a peak discharge exceeding approximately 1 000 Ml/d). Figure 9.36 shows the variation in observed flow and salinity for the Wimmera River at Upstream of Dimboola for this event. No significant salinity pulse was observed. Salinity pulses are also absent from other events.

The existing data indicate that temporary increases in salinity during mixing events do not occur routinely; however the model routinely predicts such pulses.

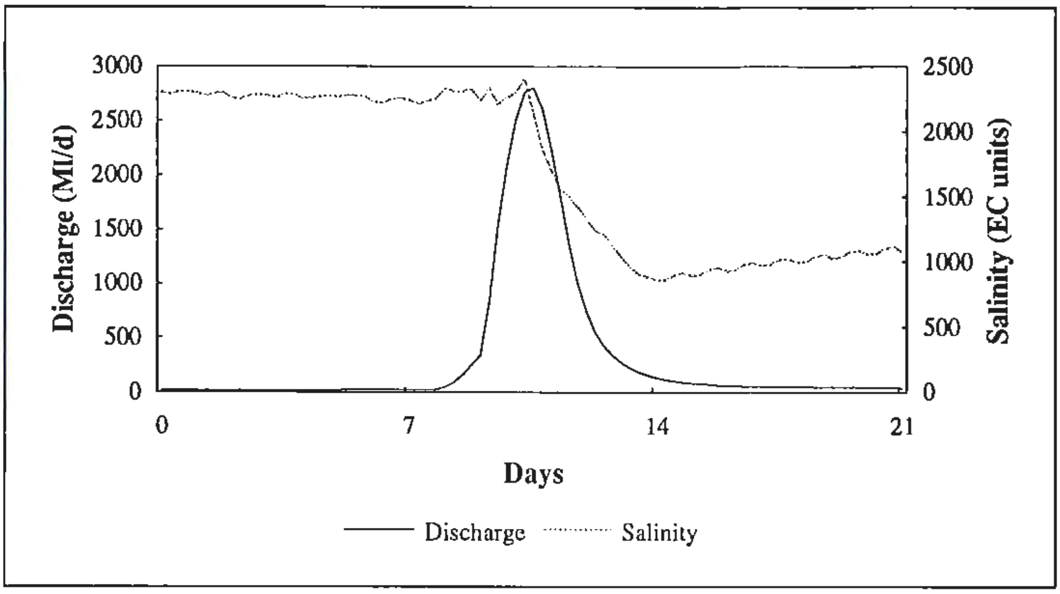


Figure 9.36: Variation in stream salinity during a mixing event for the Wimmera River at Upstream of Dimboola for the period 13/7/90 to 7/8/90.

The existence of these pulses in the simulations does not depend on the specific value of either C_F or φ nor on the rate of groundwater inflow. However it is possible that the pulses predicted by the model arise partly from representing all saline pools with a typical saline pool. While there is some variation in the time of mixing between saline pools in the model due to variations in cross-sectional area and thus velocity, the use of a typical saline pool may concentrate the flushing into an artificially short period of time. If spatial variation in C_F, φ and the scour depression geometry were included some saline pools would tend to mix at smaller discharges and others at larger discharges. Therefore the mixing would occur over a longer proportion of the hydrograph. This suggests that the amplitude of the pulse would be decreased, the length of it would be increased and it would become less noticeable.

Another plausible explanation for the absence of observed salinity pulses relates to φ. The simulated salinity pulses increase as φ or the proportion of groundwater routed through the saline layer increases. It is possible that only a small proportion of the groundwater entering the Wimmera River is routed through saline pools and as a result salinity pulses are not observed. However simulations with $\varphi = 0.25$ still indicate that salinity pulses occur. Certainly saline pools only occur in some scour pools in the stream which may indicate that only a small proportion of the groundwater is routed through saline pools; however it may simply indicate that groundwater inflows have a high spatial variability.

9.8 SUMMARY.

Two models of saline pools have been developed. Salipool is a model of individual saline pools and MIKE 11-SP is a river model which includes the effects of saline pools. Both these models describe saline pools which form as a result of groundwater inflows and which are mixed by fresh overflows. Other formation and mixing processes are not included. Saline pools are described using a layer of fresh water and a layer of saline water. Mixing of saline pools is described as a function of Ri_L and formation is described using a time-series of groundwater inflows.

Salipool was applied to four saline pools in the Wimmera River. Appropriate values of the flushing coefficient, C_F, were obtained by calibration. A generalised calibration based on bend sharpness was suggested. Calibrated values of C_F support the hypothesis that bends have an important influence on the mixing of saline pools due to their effect on the vertical velocity profile and the direction of the near bed currents.

Simulations of the seasonal behaviour of saline pools using Salipool show that the model is capable of reproducing the seasonal pattern of stratification. Errors in the predicted stratification of between -50% and +100% occur. These simulations indicate that, under the current flow regime, the behaviour of saline pools in the Wimmera River can be divided into periods dominated by formation during low flows and periods dominated by mixing during flow events.

A sensitivity analysis of Salipool was conducted. This analysis suggests that the time taken to flush a saline pool is most strongly influenced by the rate at which the discharge increases in the river. Predictions of flushing are insensitive to the buoyancy and the flushing coefficient. Predictions of the flow rate at which stratification begins to develop agree with field observations and are insensitive to the buoyancy, groundwater inflow rate and the flushing coefficient. Simulations of saline pool behaviour become sensitive when an intermediate river flow is sustained. Under these conditions a balance between flushing and the groundwater inflow develops.

Salipool was used to make predictions of stratification under a simple environmental flow scenario in which a minimum discharge for the Wimmera River at Horsham was specified. The effect of the environmental flow varied significantly between sites. Furthermore the simulations suggest that significant benefits could be gained from small increases in discharge under certain circumstances. It should be noted that Salipool has not been tested under a flow regime similar to that including the environmental flow and that predictions can be sensitive to changes in parameters and the groundwater inflow rates in these circumstances.

MIKE 11-SP was developed by modifying MIKE 11 to incorporate a description of saline pools which is based on the algorithms used in Salipool. Simulations using MIKE 11-SP indicate that saline pools reduce surface salinities during low flow periods by storing saline water. Simulations with two different estimates of groundwater inflows were conducted. These indicate that, from a modelling perspective, accurate estimates of groundwater inflow to the Wimmera River are important during low flow periods. Existing estimates appear to have significant errors associated with them. MIKE 11-SP predicts that pulses of salt associated with mixing of saline pools will occur during mixing events; however observed salinity behaviour for the Wimmera River at Upstream of Dimboola do not support this prediction.

CHAPTER 10 - DISCUSSION.

During the progress of this research a variety of issues related to stream salinity in the Wimmera River have been examined. Significant progress has been made in developing a quantitative understanding of the dynamics of saline pools and stream salinity in the Wimmera River. The previous chapters provide detailed discussions of this understanding. However this understanding is incomplete. The first part of this chapter identifies opportunities for further work which would address the major limitations of this research.

A numerical model of the Wimmera River has been developed. While much of the understanding referred to above has developed through separate experiments and analyses, this model is important in that it integrates the separate aspects of this work. Several important issues related to the practical application of sophisticated stream models of the genre of MIKE 11 have emerged during this project. These issues are likely to be important in many practical applications of models, especially when examining water quality issues at the catchment-scale. These water quality issues are becoming increasingly important.

The second part of this chapter provides a detailed discussion of these modelling issues. It is argued that the inability to describe precisely the boundary conditions and the physical characteristics (processes, channel geometry etc) of a particular stream system and to test the resulting model, are likely to be the most significant factors determining the quality of a particular modelling application. Some scale issues and the benefits of modelling are discussed. Finally, some important issues associated with generalised models are identified.

10.1 LIMITATIONS AND FUTURE RESEARCH.

This section briefly considers further research and investigation that would overcome some of the major limitations of this research or that would contribute to a greater understanding of stream systems which have features similar to the Wimmera River. Some of this work is directly related to the Wimmera River and some addresses broader questions related to specific aspects of the behaviour of natural streams that have arisen from the research. Questions specifically related to stream modelling are considered in §10.2

10.1.1 INFLUXES OF WATER AND SALT.

Both surface water and groundwater inflows contribute water and salt to the Wimmera River. Methods for estimating flow and salinity in unmonitored tributaries of the Wimmera River have been developed and used to obtain model boundary conditions. In most cases, flows have been estimated by scaling the Concongella Creek hydrograph and salinities have been estimated using solute rating curves. Inflows of groundwater to the Wimmera River between Glynwylln and Glenorchy have been estimated using a water balance approach. For the Wimmera River downstream of Roseneath, estimates of groundwater inflows were obtained from the Department of Conservation and Environment (D. Strudwick, DCE, Per. Com.).

Uncertainty in the estimates of these fluxes significantly contributes to uncertainty in the model of the Wimmera River. Improvements in the models predictive ability would be expected with further refinement of these estimates. It is believed that more significant improvements can be gained from improved estimates of groundwater influxes downstream of Roseneath since the largest simulation errors occur in the lower river during low flows; however this depends on the specific purpose for which the model is used.

Additional stream monitoring is required to refine estimates of tributary inflows. Any such stream monitoring should include monitoring of salinity at the same temporal-scale. Data from newly installed continuous salinity monitors on tributaries of the Wimmera River, including the one on Concongella Creek, should be analysed with the aim of improving salinity estimates when a sufficiently long data record becomes available. Any new research into the export of salt from catchments affected by dry-land salinity should be examined with the aim of refining estimates of salinities in tributaries of the Wimmera River. It should be noted that differences between tributaries, especially between those in the upper and lower catchment, would be expected.

Interaction between the Wimmera River and the groundwater downstream of Roseneath is an important determinant of stream salinity during low flow periods. Detailed groundwater data have been collected by the Department of Conservation and Environment, Centre for Land Protection Research and used in conjunction with flow net analyses to estimate groundwater inflows; however details of these estimates have not been published (D. Strudwick, DCE, Per. Com.). Simulations using MIKE 11 suggest that these estimates may have significant errors associated with them. These data should be reanalysed in conjunction with an analysis of data from continuous flow and salinity monitoring stations on the Wimmera River at and downstream of Horsham. This analysis should include an estimation of the uncertainty of the resulting groundwater inflow rates.

10.1.2 CHANNEL MORPHOLOGY.

Three separate aspects of stream channel morphology need to be considered. Firstly, it was shown that the Wimmera River could be divided into two statistically different morphological categories: one characterised by a single thread channel and the other by an anabranching channel. Variations between these channel types occur at a length-scale of several kilometres. It is unclear how common this phenomenon is in other natural streams and the geomorphological processes leading to the development of this large-scale quasi-periodic variation are unknown.

A methodology for characterising the along-channel variability in cross-section shape, size and vertical location has been developed for the Wimmera River. The applicability of this methodology to other streams should be investigated. If the method is generally applicable it could be used to develop an empirical understanding of typical cross-sectional variation in natural streams. The methodology should also be extended to include a quantification of the length-scales associated with channel variability.

Numerical experiments examining the hydraulic implications of channel variability have been conducted. These should be extended to a wider range of stream channels. This would allow generalisation of the conclusions drawn from this work. If such experiments were integrated with an understanding of typical stream channel variability, additional insight into the channel routing process could result.

10.1.3 WETLAND HYDROLOGY.

Simulations of flow routing in the Wimmera River indicate that off-channel storage is important even for relatively small flow events. It was hypothesised that this storage was related to the existence of backwater areas in relict channels and inundation of low lying areas in wetlands (anabranching sections). Improvements in the model of the Wimmera River could be made by developing a better description of these areas which are currently modelled using an equivalent channel.

Field visits to wetland areas indicate that there are many small channels in these areas and that the connectivity between these channels is complicated. Subtle topographic features which exist at relatively small spatial-scales are likely to be important in determining flow paths. Furthermore, it is possible that a significant proportion of the flow through these areas occurs outside identifiable stream channels. Significant wetland (eg. Barmah Forest on the Murray River (Bren et al., 1988)) and anabranching channel systems (eg. Cooper Creek (Knighton and Nanson, 1994) occur on other Australian streams. Results of hydraulic and hydrologic research into these systems are likely to contribute to the understanding of their ecology as well as their hydraulics and hydrology.

10.1.4 MIXING OF SALINE POOLS.

10.1.4.1 Flushing by Fresh Overflows.

An empirical understanding of the flushing of saline pools has been developed during this project. It was suggested that, provided bend sharpness is accounted for, the flushing can be predicted as a function of a Richardson number, Ri_{LS}. However further testing of the proposed relationship is required.

The empirical flushing relationship referred to above suggests that bends have a significant impact on saline pool flushing. The hypothesis that the effect of bends on the vertical velocity profile and the direction of near-bed currents is responsible for this effect was advanced. A series of laboratory experiments is required to test this hypothesis.

A qualitative understanding the mechanism involved in the flushing of saline pools by fresh overflows has also been developed. A thin continuous outflow of saline water up the downstream slope of the scour depression is responsible for most of the flushing, although turbulent entrainment also contributes. The saline water in the outflow consists of a mixture of the saline water from the scour depression and water from the fresh overflow. Shear stresses associated with the overflow apparently supply energy to the outflow. Surges in the outflow indicate that interfacial waves may also contribute some energy to the outflow. Further research is required to develop a quantitative understanding of these mechanisms.

10.1.4.2 Convective Mixing.

Field observations suggest that penetrative convection associated with surface cooling is responsible for mixing of saline pools on some occasions. Further field studies are required to quantify this mixing and to establish the relative importance of convection and fresh overflows in flushing saline pools in the

Wimmera River. It is possible that convective mixing can limit the formation of stratification during the Autumn period in some circumstances. If this phenomenon were common in the Wimmera River it may be of practical importance in developing an environmental flow policy aimed at reducing the occurrence of saline pools because a seasonal variation in the environmental flow may be appropriate.

Convection is also important when considering the deoxygenation of the upper layer in the Wimmera River which occurs during summer (Anderson and Morison, 1989c). Anderson and Morison (1989c) showed that a continuous environmental flow could significantly reduce this deoxygenation. The presence of convection during the Autumn period indicates that such a flow may only be required during periods when the water temperature is stable or warming. Such a possibility should be investigated if an environmental flow is to be used to improve dissolved oxygen concentrations in the upper layer.

10.1.4.3 Predicted Salt Pulses.

MIKE 11-SP, which is a model of the river incorporating saline pools, has been developed. Simulations using this model indicate that temporary increases in salinity associated with the mixing of saline pools may occur in the Wimmera River. Field observations do not support this prediction. Two limitations exist with the current model that should be addressed. The first of these is uncertainty associated total groundwater inflows and with the proportion of groundwater entering the saline layer of saline pools. Further work is required to improve these estimates and provide more realistic inputs to the MIKE 11-SP model. The second limitation is the use of a typical saline pool to represent all saline pools. Further field information on the distribution and characteristics of individual saline pools is required to overcome this limitation. It should be noted that MIKE 11-SP integrates the effects of many saline pools and that as a result the downstream salinity behaviour is unlikely to provide a reliable test of the representation of the behaviour of specific saline pools.

10.1.5 FORMATION OF SALINE POOLS.

10.1.5.1 Primary Stratification.

Primary stratification refers to stratification that develops in a scour depression as a result of groundwater intrusion to the stream within the reach associated with that depression. Primary stratification develops at the sites studied when the stream discharge is less than 200 - 300 Ml/d. The rate at which primary stratification develops has been estimated for four locations. However, details of the mechanism by which the saline water collects in the scour depression remain unclear. At least two possibilities exist and it is possible that both may occur at a specific saline pool. The groundwater may flow into the river through the river bed in the scour depression or it may flow in through the river bed or banks at a point remote from the scour depression and then flow into the scour depression under the influence of buoyancy forces.

In the later case it is possible that the rate at which groundwater flows into the saline layer will depend on the volume of water affected by the stratification. It is also possible that an intermediate stream flow would disrupt the flow into the scour depression and prevent a saline pool forming. Thus understanding the mechanism involved in the development of primary stratification has implications for the modelling of saline pools and the development of environmental flow criteria. Further research is required to develop this understanding.

10.1.5.2 Secondary Stratification.

Secondary stratification refers to the development of stratification due to saline water (and in some cases fresh water) that has flowed some distance along the stream. It may occur as a result of a buoyant inflow entering the pool from upstream and flowing into the scour depression (saline water) or over the surface of the pool (fresh water). Although it is not certain, this mechanism is thought to be relatively unimportant in the Wimmera River. A large amount of literature related to density currents and buoyant inflows to lakes and reservoirs exists which if combined with field studies could provided valuable insight into these processes.

Secondary stratification may also occur as a result of saline water overflowing from a neighbouring scour depression. Such interaction between neighbouring scour depressions probably occurs at two and possibly three of the five saline pools considered in detail. It may therefore be a relatively widespread phenomenon, especially during longer low flow periods. Field studies of sites where this interaction occurs combined with further model development are required to understand this process. It should be noted that one of the major limitations of Salipool and MIKE 11-SP is that interaction between neighbouring scour depressions is not taken into account.

10.2 DISCUSSION OF STREAM MODELLING.

A model of the Wimmera River System between Glynwylln and Glenorchy has been developed, calibrated and tested. MIKE 11 which is a numerical model based on the St Venant and Advection-Dispersion Equations was used. The model is capable of simulating flows and salinities relatively well, especially in the upper river. Results from the different parts of this research have been integrated successfully using the model. This model currently represents the most advanced quantitative understanding of salt transport in the Wimmera River. From a technical management perspective, the model provides a greatly improved basis for studying the impact of changed flow management practices on stream salinity in the Wimmera River.

While the modelling has been successful, a number of limitations do exist. These are now discussed in detail. The following discussion concentrates on the hydrodynamic modelling; however the comments are also applicable, in a general sense, to the solute transport modelling. It should be noted that MIKE 11 is typical of state-of-the-art one-dimensional computational hydraulic models and that the following comments apply to all models of this genre. The following discussion may seem self-evident, or even pedantic, in places. However the issues discussed are important from a practical modelling perspective. It is believed that the issues discussed below are frequently encountered in practical model applications and that they are often of primary importance in limiting the reliability of the resulting model.

In the following discussion it is important to remember that approximately 200 km of stream have been modelled, which represents over half of the length main stream in this river basin. The model must therefore be seen as a large-scale model. This model integrates an understanding of several different aspects of the Wimmera River developed during the progress of this research. It was developed with the ultimate aims of investigating the behaviour of salinity in the Wimmera River and of examining the implications of various flow management strategies on stream salinity. The model had to be capable of modelling individual events and therefore needed to operate on a time-step that was smaller than the event time-scale. In practice this implied that the time-step could not exceed 3 hours during events; however it could have been significantly longer for large parts of the year.

10.2.1 DATA LIMITATIONS.

In any project there is always a limitation on the amount of information that is available, the amount of field data collection that is possible and the length of time available to complete the project. These limitations may significantly restrict the methodologies that can be applied to a problem and the confidence that can be placed in conclusions drawn from specific analyses. Additional data in a number of areas such as, channel morphology, stream-aquifer interaction, tributary discharges and salinities, and additional calibration data, would certainly have assisted this project. Such information could not be collected since all the resources available for additional data collection were required for field studies of the behaviour of saline pools and for monitoring of river salinities which were even more critical to the success of the project. Even though a significant amount of additional data has been collected, data limitations have still had a significant impact on this study.

The data limitations that have been most significant in this project fall into two groups. Firstly, the boundary conditions used in the model had to be estimated in many cases. Secondly, the physical definition of the river channel required many cross-sections to be generated, anabranching channels and the Huddlestons Weir diversion to be represented in a very simplified manner and inundated areas to be estimated through calibration. This lack of data has placed a limit on the degree to which the model could be calibrated and tested and on the model complexity.

The major problem with calibration and with testing the representation of the physical system comes from the difficulty in identifying the sources of errors. While there is little doubt that there are errors in the representation of the stream, many of the errors in simulated discharges and salinities are related to poor boundary conditions. The problem is that errors in the simulation are expected even if the specification and parameterisation of the system are completely correct. Quantifying the relative contribution of poor system specification and poor boundary conditions to simulation inaccuracies is difficult.

Similarly it is not possible to be certain whether parameter "refinements" are correcting an erroneous parameter or simply compensating for some other error. The parameterisation of flow resistance is a case in point. Numerical experiments described in §5.5 show that the stage-discharge relationship at a given location is sensitive to the downstream cross-sections. If the cross-sections downstream of a gauging station are incorrectly specified, calibrations based on simulated stage will tend to compensate for this error by introducing an error into the flow resistance.

The use of stage data collected at gauging stations to calibrate flow resistance also has the potential to bias calibrated flow resistance. Gauging station locations are selected so that a good control section is available and so that vehicular access is possible. Also, control sections are cleared of debris. Therefore gauging stations are not necessarily representative of the entire stream. This implies that flow friction parameters calibrated to reproduce stage-discharge relationships at gauging stations might not be representative of the entire stream.

10.2.2 CONCEPTUALISATION AND PHYSICALLY BASED MODELLING.

One of the purported advantages of using models which solve the St Venant Equations for unsteady, gradually varied flow is their physical basis. In other words the model solves the partial differential equations that describe the physical system. However when these models are applied to systems where there are limited data available, the simplifications that prove necessary often compromise that physical basis.

In this study it has been necessary to make a number of simplifications to the representation of the system because of insufficient information on the physical system. There is no doubt that these conceptualisations have contributed to errors in model simulations. The most significant conceptualisations relate to the modelling of anabranching sections of the stream, off-channel storage and the Huddlestons Weir diversion.

For example, wetland areas are modelled by using an effective channel rather than by including individual anabranches. While it is argued that this approach is a reasonable representation of the system's behaviour, the physics of the system is not modelled in detail. This is not a criticism of MIKE 11 as such, rather it is a demonstration of the fact that limited information can force the modeller to make significant simplifications in the representation of the system. Given sufficient data and computing power, a model which incorporates detailed network descriptions in wetland areas could be developed. However such a model would require a large amount of data describing the channel network and flood plain and a significant number of stage and discharge monitoring locations within the wetland areas to test the model adequately. It is unlikely that the improvement in model performance would justify the additional costs involved.

10.2.3 PROBLEMS OF SCALE.

10.2.3.1 Scale of Description.

The primary objective of this project was to understand the processes controlling the behaviour of stream salinities in the Wimmera River so that ultimately the impact of different flow management practices could be assessed. A major issue was the behaviour of saline pools under different flow regimes. A drainage basin-scale model is required to meet the first objective, yet it must include features which are characterised by length-scales of less than 100 m.

A practical lower limit for the average cross-section spacing in MIKE 11 for this project was approximately 1 000 m which resulted in a time-step limited by instabilities to 15 minutes or less. Simulation of flow and salinity for 1 year then requires approximately 2 hours of CPU time on a Hewlett Packard 9000-720 workstation. Given that the model is ultimately for planning purposes and had to be capable of running simulations for periods of several years for a variety of management options, to reduce grid separations any further was impractical. Yet to resolve individual saline pools and thus make full use of the model's physical basis the grid separations would have to be reduced by more than an order of magnitude. Saline pools were built into the model of the Wimmera River by using a conceptual model of saline pool processes and averaging the storage of saline water over a grid cell (~500 m in the salinity simulations).

A similar problem exists if modelling wetlands with a network model. In this case a significant number of channels are much shorter than 1 km. Modelling these areas would require much more detail than that provided with channel cross-sections specified every 1 km.

This scale problem is frequently encountered when modelling natural systems. Examples include description of sub-grid variability in physically based rainfall-runoff models (Beven, 1989) and description of energy loss processes in open channel flow. Losses in irregular channels depend on a number of factors including channel substrate, bed forms, bends, secondary currents, large woody debris and expansion and contraction losses (French, 1986) which are characterised by a variety of length-scales. This amalgam of processes is typically described by lumping them together and averaging which allows the application of one of the resistance equations (Weinmann and Laurenson, 1979) developed for uniform steady flow such as Manning's equation. Indeed even Manning's Equation can be thought of as a conceptual model which averages smaller scale turbulent process associated with energy dissipation.

It should be noted that algorithms developed for one process or at one scale may not be an appropriate description for another process or at a different scale. For example, use of the one-dimensional Advection-Dispersion equation for prediction of solute transport is not appropriate where significant transient storage exists, because the one-dimensional flow assumptions are violated (Bencala and Walters, 1983).

When used appropriately, averaging and conceptualisation is a practical and powerful approach to overcoming the problem of small-scale variability. However when algorithms developed for one process are used for some analogous process or group of processes, the parameters involved tend to lose their physical meaning. This is the case when Manning's Equation is used to describe energy losses in natural channels and is the case in many other models. As Beven (1989) argues for the case of surface hydrology models, the so called physically based models are really "lumped conceptual models" at the grid-scale. It is obvious that issues of model-scale and conceptualisation are fundamentally linked.

10.2.3.2 Scale of Testing.

In many modelling studies the scale at which model predictions can be tested is significantly greater than the scale at which models are discretised. This often occurs when the modeller wishes to predict the detailed spatial behaviour of the system. In that case, the scale at which data for model testing is available is especially important. Measured data usually represent some form of integration or averaging. In the case of travel time between two gauging stations the storage processes between the two gauges are averaged. This means testing of routing simulations can only be carried out on a spatial-scale equal to the separation of the gauging stations. This is usually significantly larger than the grid spacing which is the scale at which the model makes predictions. In the Wimmera River, fully rated gauging stations are separated by 48 km on average. The reaches between gauging stations include sections of stream with a single channel and anabranching sections. Therefore, it has not even been possible to test the predicted wave celerity in each of these classes of channel.

The numerical experiments described in §5.5 demonstrate the danger of using data representing a large-scale integration to test a model when local predictions are important. Taking residence time (or velocity) as an example, changes in the variability of the cross-section width had almost no effect when averaged over the reach yet significant changes occurred at individual cross-sections. The mean residence time (or velocity) calculated from a dispersion test would have been similar in each case even though the local residence times were quite different. The point is that a well simulated outflow hydrograph does not imply that velocities and water levels at specific internal computational points are predicted correctly.

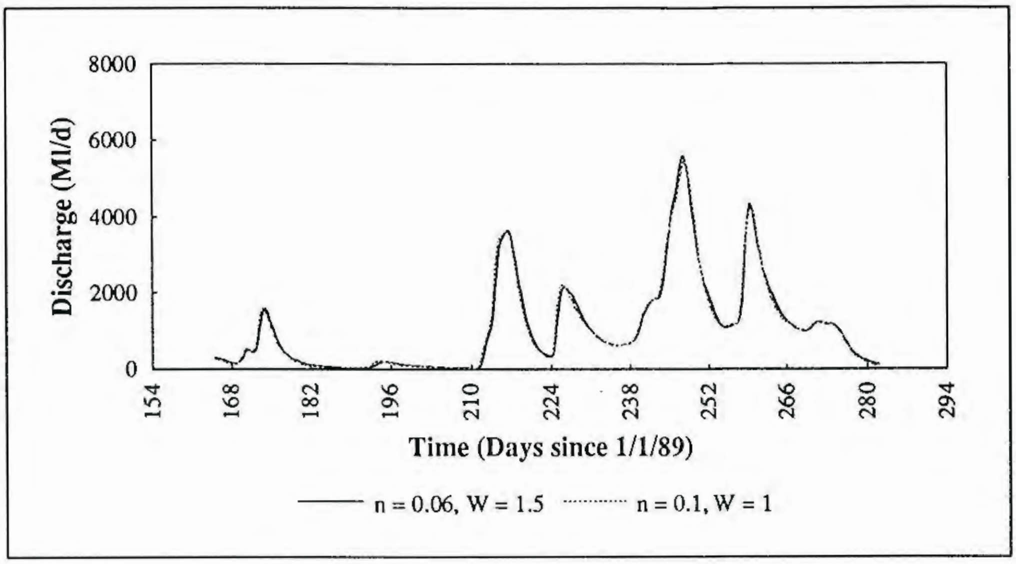


Figure 10.1: Comparison of simulated hydrographs for the Wimmera River at Lochiel with two different model parameterisations.

10.2.4 PARAMETER IDENTIFICATION.

Figure 10.1 shows two hydrographs that were simulated for the Lochiel gauging station using two different models of the Wimmera River. The first is that simulated using the model as calibrated and the second is from a model which does not include any additional storage but has had the flow resistance increased significantly. Essentially the storage has been increased in both models but by using different approaches. There is little difference between the simulations especially given that the flow resistance was adjusted by increments of 0.01 which is 17% of the calibrated value. Finer adjustment of flow resistance would have improved the agreement between the two simulations.

Several points can be made about this. Firstly, there is obviously a significant parameter identification problem if a hydrodynamic model including both these parameters has been calibrated on flood wave celerity alone. This problem was overcome in this instance by using rating curves to calibrate for flow resistance and then the wave celerity to calibrate for additional storage. It could be argued that this is a problem specifically related to this project since additional storage would not normally be considered a calibration parameter. That is a justifiable criticism and demonstrates the problems of using hydrodynamic models in situations where there are insufficient data. However there is a more subtle point to make in relation to model testing and the interpretation of the flow resistance parameter.

Figure 10.1 demonstrates that there is an intimate link between the amount of storage (channel + flood plain) included in a model, the flood wave celerity and the flow resistance. This implies that, when the flow resistance is calibrated to reproduce wave celerity, the calibrated value will reflect both the actual average flow resistance and errors in specification of cross-sections and inundated areas. Similarly it was argued in §6.3.1.1 that, if the calibration concentrates on the prediction of stage, the calibrated flow resistance will reflect both the downstream flow resistance and any errors in the downstream channel specification. These problems also affect the testing or verification stage of the modelling process. Therefore, calibration and verification does not ensure that physically correct parameter values have been obtained.

10.2.5 THE BENEFITS OF MODELLING.

There are a number of reasons for using simulation models. Primarily models provide a tool for critical analysis (Konikow and Bredehoeft, 1992). Modelling studies often provide analyses which contribute to a decision making process. Those decisions may be policy or planning decisions, design decisions or real time system operation decisions. In other cases the predictions may be made in a research context and are primarily made for the purposes of exploring system behaviour and hypothesis testing (Konikow and Bredehoeft, 1992; Grayson et al., 1992). The process of model development can also lead to new insights into system behaviour (Hillel, 1986; Konikow and Bredehoeft, 1992; Grayson et al., 1992). Models also have another equally important role. That is, they provide a theoretical framework within which the research or investigation can be carried out (Hillel, 1986; Grayson et al., 1992).

In this study the model has been very valuable in its role of providing a theoretical framework. This has allowed disparate pieces of information, especially those relating to tributary inflows and salinities, to be collected and integrated in a consistent manner. In situations such as this where information is limited this organisational role becomes even more important because of the need to obtain maximum benefit from all the available information. The model has also provided a tool which is capable of simulating in-bank flows and the general behaviour of salinity over time. This will be valuable for exploring the sensitivity of the system to different flow management practices and thus providing information which can be used to better manage the Wimmera River.

The model development process has also indicated that there is a significant source of additional storage which is influencing the routing of hydrographs down the Wimmera River. Unfortunately due to the limited data available there is a number of alternative hypotheses which could explain the routing behaviour of the system. Therefore, while it has been hypothesised that this additional source of storage is related to relict channels, anabranches and low lying areas in wetlands, the lack of data has prevented proper testing of this hypothesis.

10.2.6 A NOTE ON GENERAL MODELS.

A problem that can be encountered when using general models is that it may not be possible to describe adequately a particular feature of the system with the available algorithms. For example, the MIKE 11 model had to be modified to simulate the Huddlestons Weir Diversion and the effect of saline pools. The model was also unable to model low flows using a standard channel reach. Both these problems were overcome; the first by writing additional subroutines and the second by introducing fictitious weirs. If it had not been possible to add new algorithms to the model, the modelling of Huddlestons Weir would have been significantly more difficult and it would not have been possible to model saline pools. Thus the ability to add code to a generalised model is a significant advantage and can overcome the situation where a feature has to be modelled in an inadequate manner. Sufficient mathematical and programming skill and a deep understanding of the model algorithms are still required.

A more serious problem arises when there is a feature designed to automatically handle some numerical difficulty such as the simulation of low flows. It is often not possible to disable these features and it is difficult to change the algorithms used to handle the problem unless, or probably even if, the full code is available. Writing a model for a specific job would allow more flexibility in the algorithms used and, for systems with some unusual features, is a viable alternative; especially since experienced modellers could develop a toolbox of algorithms for different processes (Grayson and Nathan, 1993).

Another problem with the large generalised modelling systems is that they tend to be relatively difficult to understand fully and thus apply in a rigorous manner. Examples from MIKE 11 include the calculation of energy losses in drowned weir flow, the simulation of low flows and the way additional (off-channel) storage is included in solute transport calculations. In the last case the only way to obtain information on the algorithms was to contact the Danish Hydraulics Institute. Floyd and Cox (1993) have also identified the above limitations with the current state of the art generalised models.

In contrast to the difficulties in understanding the subtleties of the algorithms incorporated in the models, vast improvements in user interfaces have meant that actually setting up a model and getting simulations to run is becoming relatively straightforward. This means that it is possible for a person who is unfamiliar with computational hydraulics and the physical processes being simulated to obtain results based on incorrect or inadequate assumptions. One of the most important steps in any modelling exercise is the conceptualisation of the system. This requires a proper understanding of the processes likely to be operating and of their relative importance. However with an array of sophisticated algorithms available it is tempting to simply use them rather than to assess the importance of different processes and then choose appropriate algorithms. This amounts to a substitution of apparent sophistication for detailed understanding (Grayson and Nathan, 1993). Cunge et al. (1980) also express concern about the black-box use of river modelling systems. Overcoming this potential for abuse of the modelling technology is a dilemma that model developers, practitioners and educators must face as a matter of priority.

10.3 SUMMARY.

The first part of this chapter discusses issues requiring further research and investigation that have arisen during this project and that are considered to have important implications. These include issues associated with, the estimation of influxes of water and salt to the Wimmera River, stream channel morphology, wetland hydraulics and hydrology, and the formation and flushing of saline pools. The second part of the chapter is a discussion of some important issues related to the large-scale modelling of natural streams.

Many of the limitations of the model of the Wimmera River are related to the limited data available relating to the physical system and to the system's inflows and outflows. Limited data on the physical system, particularly on channel morphology in anabranching areas and on the system in the vicinity of Huddlestons Weir, has resulted in the MIKE 11 model being applied in a conceptual manner. Calibration and testing of the model have been limited by errors associated with estimation of data for ungauged tributaries.

Issues of scale and the description of the physical system and scale and model testing have also been discussed. Because of relatively large gauging station separations compared with the model grid-scale it has not been possible to test detailed model predictions. Even though these limitations exist, the model has provided an invaluable theoretical framework for this study. The model represents the quantitative understanding of the behaviour of salinity in the Wimmera River developed during this project and it is a valuable tool that can be used to improve the management of the Wimmera River.

CHAPTER 11 - SUMMARY AND CONCLUSIONS.

Land and water salinisation in parts of Australia has expanded dramatically since European colonisation. The primary cause of this expansion has been a significant change in the hydrologic balance due to changes in vegetative systems, notably the replacement of native vegetation with shallow rooted crops and pastures, and the advent of irrigation. Stream salinisation is recognised as a significant problem from both the water supply and the environmental perspectives. The research described in this thesis relates to the Wimmera River which is located in north-western Victoria and is severely affected by salinisation.

The Wimmera River catchment is predominantly used for agriculture. Climate in the catchment varies from sub-humid in the ranges of the south to semi-arid in the north. The Wimmera River rises in the Pyrenees Ranges and flows north-west to Horsham collecting water from tributaries flowing from both the Pyrenees and Grampians Ranges along the way. Downstream of Horsham the Wimmera River turns and flows north to Lake Hindmarsh which is the first of a series of terminal lakes. In their head-waters, the major tributaries of the Wimmera River are perennial upland streams. Further downstream the Wimmera River becomes an ephemeral stream which contracts to a series of large pools during the summer. The flow regime in the Wimmera River is highly variable and seasonal. Flows predominantly occur in late winter and spring. Water is diverted from the Wimmera River to the Wimmera Mallee Stock and Domestic Water Supply System (WMSDS) at Glenorchy and at Huddlestons Weir.

Salinities in the Wimmera River are relatively high and are inversely related to discharge. Salt in the Wimmera River originates primarily from surface runoff generated in salinised parts of the upper catchment and from groundwater flowing into the river in the upper catchment and downstream of Roseneath. During low flow periods saline groundwater collects in scour depressions forming a series of density stratified or saline pools in the lower river. The saline water in these pools is typically anoxic, has a low pH and contains hydrogen sulphide. This renders the water toxic to fish and other aerobic aquatic organisms.

At the start of this project there was significant community concern about high salinities in the WMSDS and in the Wimmera River and about the environmental effects of saline pools. There was also concern about the lack of environmental flow provision to the Wimmera River. The Rural Water Corporation, who operate the WMSDS, were concerned and wished to be able to quantify the effects of different flow management policies on salinities in the Wimmera River. There was a clear need to develop a quantitative understanding of the processes controlling the behaviour of salinity in the Wimmera River so that the effects of changed management strategies could be predicted.

This thesis describes research on the Wimmera River that has been conducted with the aim of developing the above understanding. A numerical model of the Wimmera River was developed to integrate the results of the various analyses, to provide a theoretical framework for hypothesis testing and to make the above predictions. MIKE 11 was used to simulate the Wimmera River.

At the beginning of the project it was known that the majority of flow in the Wimmera River originated from runoff during storms in the catchment. Furthermore the runoff events associated with these storms were known to cause rapid water quality changes in the lower river. Given the importance of tributary inflows and the fact that many tributaries were ungauged, it was necessary to estimate the flows and salinities for these streams. With the exception of the McKenzie River, flows from ungauged tributaries were estimated by scaling the flow in Concongella Creek by the ratio of the mean annual flow in Concongella Creek and the ungauged stream. A regression relationship between catchment area and mean annual runoff was developed and used to obtain the scaling ratio. For periods when gauged flows were unavailable, flows in the McKenzie River were estimated using a regression relationship with discharge in Burnt Creek.

The salinities of all surface water inflows had to be estimated. It was found that a solute rating curve estimated using spot salinity samples and the minimum variance unbiased estimator for log-linear regressions provided the most accurate salinity estimates for the Wimmera River at Glynwylln. For other tributary inflows a log-linear solute rating curve was estimated and used to predict stream salinities. The slope of these rating curves was assumed to be -0.3 which was found to be consistent with similar unregulated streams in the catchment. To estimate the coefficient for the log-linear rating curve for most streams, it was assumed that the annual salt flux per unit catchment area was 14.5 $t/a.km^2$ which is the median salt flux for gauged unregulated streams in the Wimmera River catchment. For Norton and Daragan Creeks the coefficient of the solute rating curve was estimated used measured salinities.

Groundwater inflows are also an important source of salt in the Wimmera River. The rate of groundwater inflow to the Wimmera River between Glynwylln and Glenorchy during low flow periods was studied using water and salt balances.

The mean groundwater inflow rate based on a water balance during low flow periods between the 1/9/1976 and the 31/8/1990 was 2.4 Ml/d. This is an order of magnitude less than an estimate based on a 13 year average water and salt balance conducted by Hooke (1991). It is believed that the errors in Hooke's study are significantly larger because: flows from the WMSDS were included; high flow periods were included; and both flow and salt balances were required to make the estimate. A flow rate of 2.4 Ml/d was used to specify groundwater inflows to the Wimmera River between Glynwylln and Glenorchy in the model of the River. Estimates of groundwater inflows to the Wimmera River downstream of Roseneath were obtained from a study by the Department of Conservation and Environment, Centre for Land Protection Research (D. Strudwick, DCE, Per. Com.).

In a hydrodynamic model the specification of the river system is important. This involves defining the channel network and specifying cross-sections in each channel. Estimates of the volume of water contained in large pools in the Wimmera River were significant compared with the total volume of water in intermediate flow events. Therefore inclusion of these large pools, which result from channel variability, in the model was considered important. Furthermore a significant number of cross-sections required infilling.

The morphology of the Wimmera River channel was studied in detail. A methodology for studying the along-channel variability in cross-sectional characteristics was developed. The shape of individual cross-sections was characterised using a power curve relating width to elevation. The vertical position of the cross-section was characterised using the displacement of the invert from the mean grade line and the bank-full depth characterised the vertical extent of the cross-section. It was shown that two statistically different channel types exist in the Wimmera River: one characterised by a single channel and the other by anabranching channels. Reaches characterised by one channel type have a typical length-scale of several kilometres. Longitudinal trends in channel properties were examined. As one moves downstream:

- the channel becomes shallower and the invert more variable;
- the width to depth ratio increases;
- the bank-full cross-sectional area is approximately constant;
- the average slope decreases;
- the cross-sections become less V and more U shaped.

The results of the above analysis were used to develop a stochastic cross-section model which was used to infill cross-sections for the model of the Wimmera River.

A series of numerical experiments were conducted which examined the hydraulic implications of channel variability in a stream typical of the Wimmera River. These experiments showed that, except at low flows, the depth, stage and residence time were insensitive to changes in channel variability when averaged over a 100 km long reach. However the response at individual cross-sections was sensitive to the same changes. Flood wave celerity and attenuation were insensitive to changes in channel variability. Flow friction was more important than channel variability in determining routing behaviour. These results show that including channel variability in a hydraulic model is important whenever low flows or detailed (ie at individual grid points) predictions are important. They also imply that data representing an integration of processes, such as routed hydrographs, are insufficient for proper testing of internal model predictions.

A one-dimensional model of the Wimmera River between Glynwylln and Lochiel was developed using MIKE 11 and the results from the above analyses. This model is applicable to in-bank flows and their associated salinities. A significant number of tributary inflows and salinities have been estimated and cross-sections infilled for this model. Furthermore simplifications in the representation of anabranching sections and the Huddlestons Weir diversion were necessary. It was necessary to include a series of weirs in the river so that the model simulated low flows adequately.

The flow resistance parameter was calibrated so that rating curves at each gauging station were simulated adequately. With the exception of water supply channels, a single value of the flow resistance parameter was used at all locations and flows. After calibration of the flow resistance parameter, a significant error in the flood wave celerity existed. This error is thought to be related to storage of water in relict channels and low lying areas in wetlands. An additional parameter which increased the storage width of the channel was added to the model and calibrated.

Predictions of discharge at Glenorchy are good. The quality of flow predictions at gauging stations further downstream is variable between events but consistent between stations for a given event. There is a tendency for the model to under-predict both the peak and total discharge at these gauging stations and the wave celerity is over-predicted for large events. It is likely that many of the errors in the discharge predictions are attributable to boundary condition errors; however the crude manner in which additional storage has been incorporated, the simplifications in the representation of wetland areas and the exclusion of the flood plain also contribute to the error.

The solute transport model predicts salinities well during moderate and high flow periods; however significant errors occur during low flow periods in the lower river. Errors in the predictions of low flows, in the specification of groundwater inflows, in the volume of water stored in the channel and the exclusion of saline pools contribute to these errors. Transport of salt down the Wimmera River is dominated by moderate and high flows.

After the one-dimensional model has been discussed, the emphasis of the thesis changes to a consideration of saline pools. The relevant literature dealing with stratified flow is reviewed. In particular, a study of the flushing of saline water from a square cavity is discussed and its limitations from the perspective of understanding the flushing of saline pools are identified.

A series of field and laboratory investigations into the behaviour of saline pools was conducted. Data collected as part of this and other studies indicate that most saline pools develop during low flow periods and are mixed by significant flow events. This behaviour is typical of saline pools which form as a result of saline groundwater inflows and which are flushed by fresh overflows. The seasonal behaviour of saline pools tends to be dominated by the seasonal behaviour of the stream discharge. This seasonal behaviour is complicated at some saline pools by convective mixing and formation of stratification as a result of saline river inflows. Relatively fresh river inflows were also observed to result in stratification. Longitudinal surveys of saline pools conducted during this research showed that approximately half the scour depressions in the reaches surveyed were stratified and that approximately one quarter of the length of the river was affected during these surveys.

Initial investigations of the flushing of saline pools by fresh overflows indicated that turbulent entrainment could not account for the observed mixing. Therefore a series of laboratory experiments was initiated. These were conducted by Nolan (1994) and indicated that the flushing of saline pools was dominated by a thin layer of saline water flowing up the downstream slope of the depression. Presumably this saline water is mixed turbulently downstream. It was argued that this flushing process could be parameterised using a Richardson number (Ri_{LS}) which represents the ratio of the buoyancy and inertia forces resolved along the downstream slope.

A comparison of flushing observed in the laboratory and the field indicated that mixing was significantly greater in the field for a given value of Ri_{LS}. It was hypothesised that the effect of bends on the vertical velocity profile and the direction of near-bed currents was responsible for these differences. Subsequent modelling of saline pools showed that a mixing relationship which accounted for bend sharpness could explain the observed results.

Field data showed that fresh overflows were not the only process responsible for mixing saline pools. Mixing associated with surface cooling also occurs in some locations. Examination of the seasonal pattern of water temperature variations indicates that this convective mixing is an annual phenomenon. The relative importance of mixing by fresh overflows and mixing due to surface cooling is unclear.

The formation of saline pools due to groundwater inflows was examined at four locations. Saline pools begin to develop when the stream discharge decreases to between 200 Ml/d and 300 Ml/d. The rate of saline pool formation is independent of the amount of stratification at some sites but at others it apparently increases as the amount of stratification increases. Mean rates of formation at the four sites studied varied between 0.008 Ml/d and 0.021 Ml/d. At present the mechanism by which the saline water collects in the scour depression and thus forms a saline pool is unclear.

Two models of saline pools dominated by groundwater inflows and mixing due to fresh overflows have been developed. Salipool applies to individual saline pools and MIKE 11-SP incorporates the effect of a series of saline pools in the model of the Wimmera River. Saline pools are represented using a layer of fresh and layer of saline water in both models. Groundwater inflows are specified using a time-series and mixing is a function of a Richardson number (Ri_L).

Salipool was applied to four saline pools in the Wimmera River. A generalised calibration of the mixing component of the model was suggested on the basis of bend sharpness; however further testing of this relationship is required. The seasonal behaviour of the four saline pools was simulated successfully using Salipool. These simulations indicated that under the current flow regime the behaviour of saline pools in the Wimmera River could be divided into periods dominated by formation and periods dominated by mixing.

A sensitivity analysis of Salipool was conducted. The flushing of saline pools is insensitive to the model parameters. The time taken to flush a given saline pool depends on the rate at which the discharge in the river increases. Salipool predicts the discharge at which stratification appears during flow recession correctly and this prediction is insensitive to model parameters. When a continuous intermediate river flow is maintained a balance between the inflow of groundwater and the rate of flushing develops. Salipool becomes sensitive to model parameters and especially to the stream discharge and the groundwater discharge under these circumstances. The dichotomy between formation and flushing periods also disappears.

Salipool was used to predict the impact of a simple environmental flow release. Results of these simulations indicate that the impact of a particular environmental flow varies between saline pools and that, in certain circumstances, a small increase in discharge can have a significant benefit.

MIKE 11-SP was developed using Salipool as the basis for describing saline pools and was applied to the Wimmera River between Horsham and Lochiel. Simulations using MIKE 11-SP indicate that saline pools reduce surface salinities during low flow periods by storing salt and that salt pulses associated with the mixing of saline pools may occur. However field data do not show any evidence of these salt pulses and they may be an artefact of the use of a typical scour depression and mixing relationship to describe all saline pools. Alternatively only a small proportion of the inflowing groundwater may enter saline pools. Results of the MIKE 11-SP simulations indicate that the salinity in the Wimmera River downstream of Horsham is dominated by the salinity of the upstream inflow during moderate to high flow periods and by groundwater inflows during low flow periods. Uncertainty in groundwater inflows to the Wimmera River have limited the conclusions that can be drawn from the application of MIKE 11-SP.

Some important questions which have arisen during this research remain unanswered. These have been discussed in Chapter 10. Substantial improvements in the predictive ability of the MIKE 11 model are likely to result from more accurate estimates of tributary and groundwater inflows and salinities. Several issues related to stream channel morphology, the hydraulic effect of channel variability and the hydraulics and hydrology of wetlands exist. Further research into the processes responsible for the formation and flushing of saline pools is required.

Finally a number of important issues related to the application of models to practical stream water quality problems are discussed. As a result of limited data on channel morphology and the channel network near Huddlestons Weir and in wetland areas, MIKE 11 has been applied in a conceptual manner. Calibration and testing of the model has been limited by errors in estimated inflows and inflow salinities. Calibrated model parameters are likely to reflect these errors to some degree. Important issues related to the scale at which the physical system can be specified and the scale at which the model can be tested also exist. Because of the relatively large separation between gauging stations compared with the model grid-scale, it has not been possible to test detailed model predictions. Finally some parameter identification issues, the benefits of modelling and some issues associated with generalised models are discussed. It is believed that the above limitations exist in many applications of stream models. Nevertheless, the model provides a useful tool for exploring the impact of different flow management strategies on salinity in the Wimmera River.

Significantly improved quantitative understandings of the behaviour of saline pools and the behaviour of salinity in the Wimmera River have been developed. Two models have been developed which express this understanding: Salipool; and the MIKE 11 model. Salipool is applicable to individual saline pools and the MIKE 11 model applies to a large section of the Wimmera River. Together these models form a basis for studying the impact of changed flow management strategies on salinity in the Wimmera River, and therefore for management of a significant environmental problem.

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APPENDIX 1: MIKE 11 SOLUTION SCHEME.

Relevant details of the computations performed by MIKE 11 are provided in this Appendix. The following details have been taken from the MIKE 11 Technical Reference (DHI, 1992b). MIKE 11 solves the following basic equations for unsteady flow or simplifications thereof:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\alpha\frac{Q^2}{A}\right) + gA\frac{\partial H}{\partial x} + \frac{gn^2Q|Q|}{A^2R^{4/3}} = 0 \qquad \text{(A1.1)}$$

$$\frac{\partial Q}{\partial x} + B_{st}\frac{\partial H}{\partial t} = q. \qquad \text{(A1.2)}$$

where Q is the discharge, A is the cross-sectional area, H is the water surface elevation, R is the hydraulic radius, α is a momentum coefficient, B_{st} is the storage width and q is the lateral inflow per unit length.

A1.1 SOLVING THE COMPLETE EQUATIONS.

Figure A1.1 shows the centring of the momentum equation. The following equations are used to approximate the derivatives in Equation A1.1:

$$\frac{\partial Q}{\partial t} = \frac{Q_j^{n+1} - Q_j^n}{\Delta t} \qquad \text{(A1.3)}$$

$$\frac{\partial}{\partial x}\left(\alpha\frac{Q^2}{A}\right) = \frac{\left(\alpha\frac{Q^2}{A}\right)_{j+1}^{n+1/2} - \left(\alpha\frac{Q^2}{A}\right)_{j-1}^{n+1/2}}{\Delta 2x_j} \qquad \text{(A1.4)}$$

$$\frac{\partial H}{\partial x} = \frac{\delta\left(H_{j+1}^{n+1} + H_{j+1}^{n}\right) -(1-\delta)\left(H_{j-1}^{n+1} + H_{j-1}^{n}\right)}{\Delta 2x_j} \qquad \text{(A1.5)}$$

In Equation A1.4 the quadratic term is specified as $Q^2 = fQ_j^{n+1}Q_j^n - (f-1)Q_j^nQ_j^n$ to ensure that the correct sign for the term when the flow reverses. f was set at the default value of 1 in all simulations. δ in Equation A1.5 represents the centring of the gradient of the water surface slope in time and is set such that $0.5 \le \delta \le 1$. A dissipative effect is introduced if $\delta > 0.5$. When Equations A1.3, A1.4 and A1.5 are substituted into Equation A1.1 it can be written as:

$$\alpha_j H_{j-1}^{n+1} + \beta_j Q_j^{n+1} + \gamma_j H_{j+1}^{n+1} = \delta_j \qquad \text{(A1.6)}$$

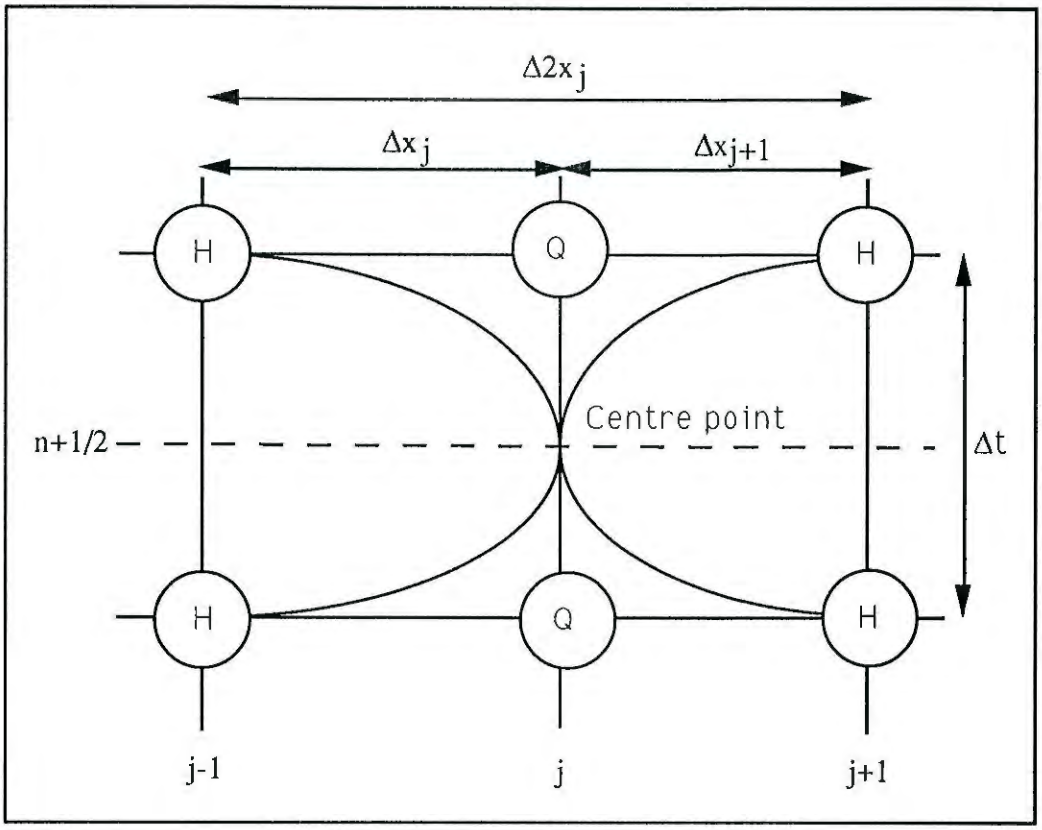

Figure A1.1: Centring of the momentum equation used in MIKE 11. After DHI (1992b).

where α_j, β_j, γ_j and δ_j are functions of the hydraulic parameters and flow and water level at time n.

Figure A1.2 shows the centring of the continuity equation. The following equations are used to approximate the derivatives in Equation A1.2:

$$\frac{\partial Q}{\partial t} = \frac{0.5\left(Q_{j+1}^{n+1} + Q_{j+1}^{n}\right) - 0.5\left(Q_{j-1}^{n+1} + Q_{j-1}^{n}\right)}{\Delta 2x_j} \qquad \text{(A1.7)}$$

$$\frac{\partial H}{\partial t} = \frac{H_j^{n+1} - H_j^{n}}{\Delta t} \qquad \text{(A1.8)}$$

In Equation A1.2 B_{st} is approximated by $(A_{0\,j} + A_{0\,j+1})/\Delta 2x_j$ where $A_{0\,j}$ and $A_{0\,j+1}$ are the plan areas of the water surface associated with Δx_j and Δx_{j+1} respectively. Substituting Equations A1.7 and A1.8 into A1.2 the continuity equation becomes:

$$\alpha_j\, Q_{j-1}^{n+1} + \beta_j\, H_j^{n+1} + \gamma_j\, Q_{j+1}^{n+1} = \delta_j\,, \qquad \text{(A1.9)}$$

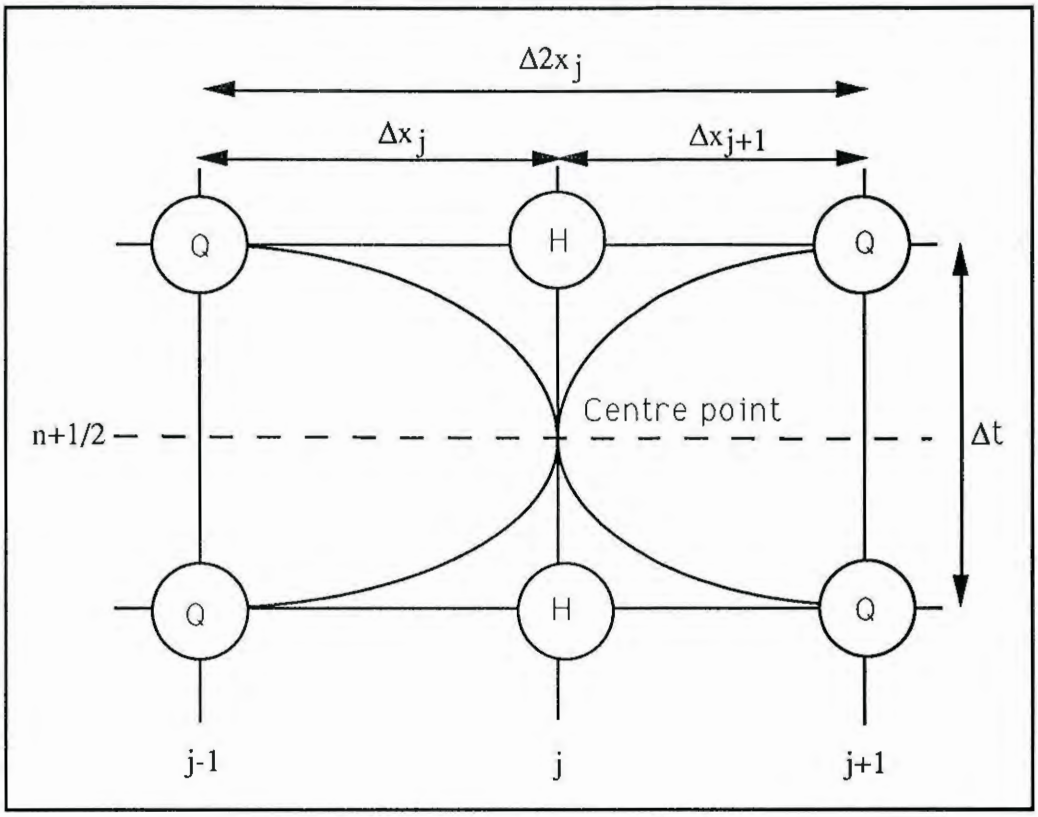

Figure A1.2: Centring of the continuity equation used in MIKE 11. After DHI (1992b).

where α_j, β_j, γ_j and δ_j are functions of the hydraulic parameters and flow and water level at time n.

For a channel reach Equations A1.6 and A1.9 form a series of simultaneous equations. Since some of the geometric variables in the coefficients are unknown at time n+1, a two step iterative solution is used. The geometry at time n is used in the first iteration. For the second iteration the geometry is calculated using results from time n and n+1.

A1.2 THE DIFFUSIVE WAVE APPROXIMATION.

With flow friction specified using Manning equation the diffusive wave approximation can be written as:

$$Q = \frac{AR^{2/3}S^{1/2}}{n}, \tag{A1.10}$$

where Q is the discharge, A is the cross-sectional area, R is the hydraulic radius, S is the water surface slope and n is Manning's n. Equation A1.10 is linearised as follows.

$$Q^{n+1} = Q^{n+1/2} + \frac{\partial Q}{\partial H}(H^{n+1} - H^{n+1/2}) \tag{A1.11}$$

$$\frac{\partial Q}{\partial H} = \frac{\partial Q}{\partial A}\frac{\partial A}{\partial H} + \frac{\partial Q}{\partial R}\frac{\partial R}{\partial H} + \frac{\partial Q}{\partial S}\frac{\partial S}{\partial H}$$

$$\approx \frac{\partial Q}{\partial A}\frac{\partial A}{\partial H} + \frac{\partial Q}{\partial S}\frac{\partial S}{\partial H} \tag{A1.12}$$

The derivatives in Equation A1.12 can be written as:

$$\frac{\partial Q}{\partial A} = \frac{R^{2/3}S^{1/2}}{n} \quad \text{and} \quad \frac{\partial A}{\partial H} = B \tag{A1.13a}$$

$$\frac{\partial Q}{\partial S} = \frac{1}{2}A\frac{R^{2/3}}{n\sqrt{S}} \quad \text{and} \quad \frac{\partial S}{\partial H} = \frac{1}{\Delta x} \tag{A1.13b}$$

Inserting Equation A1.13 into A1.12 and rearranging gives:

$$Q^{n+1} = Q^{n+1/2} + \left(\frac{B\Delta xS}{A} + \frac{1}{2}\right)\frac{AR^{2/3}}{n\Delta x\sqrt{S}}(H^{n+1} - H^{n+1/2}) \tag{A1.14}$$

Equation A1.14 forms the basis of the diffusive wave approximation. With $Q^{n+1/2}$ calculated as $(Q^{n+1} + Q^n)/2$ and $H^{n+1/2}$ calculated similarly Equation A1.14 can be rearranged to obtain Equation A1.6 and the solution proceeds as before. It should be noted that this description is centred at time n+3/4 and that some numerical damping occurs which implies that only relatively steady (compared with the time-step) backwater phenomenon are resolved. When the water surface slope becomes less than 0.0001 this solution is altered such that it becomes completely upstream centred.

A1.3 STRUCTURES.

Hydraulic structures are included in MIKE 11 by substituting the dynamic equation (Equation A1.1 or A1.10) at a specific Q-point with an appropriate mathematical description of the structure. The case of broad-crested weirs is considered here.

Three flow conditions may exist at a broad crested weir: zero flow; critical flow (undrowned); and drowned flow. MIKE 11 determines the flow condition as follows. If the upstream and downstream water surface elevations are less than the invert level of the structure the flow is zero. This is represented by setting the coefficients in Equation A1.6 such that $\alpha_j = 0$, $\beta_j = 1$, $\gamma_j = 0$ and $\delta_j = 0$. To distinguish between undrowned and drowned flow the energy at the structure is compared with the energy downstream of the structure including an allowance for an outflow energy loss. When the energy at the structure is the same as or greater than that downstream of the structure the flow will be undrowned, otherwise the weir will be drowned. That is, undrowned flow occurs when:

$$H_s + (1 - \zeta_2)\frac{Q_s^2}{2gA_s^2} \geq H_2 + \frac{Q_s^2}{2gA_2^2}, \qquad \text{(A1.15)}$$

where Q_s is the discharge through the structure, A is the area of flow, ζ_2 is the outflow energy loss coefficient and the subscripts s and 2 refer to points at and downstream of the structure respectively. During drowned flow the structure is represented using the energy principle. The head loss over the structure is:

$$\left(H + \frac{Q^2}{2gA^2}\right)_{j+1} - \left(H + \frac{Q^2}{2gA^2}\right)_{j-1} = \zeta\frac{|Q_s|Q_s}{2gA_s^2}, \qquad \text{(A1.16)}$$

where $\zeta = \zeta_1 + \zeta_2$ is an energy loss coefficient equal to the sum of the inflow and outflow energy loss coefficients. By collecting the velocity heads and expressing the equation at time n+1 the following may be obtained:

$$Q_s^{n+1} = \left(H_{j-1}^{n+1} - H_{j+1}^{n+1}\right)\sqrt{\frac{2g}{\left(H_{j-1}^{n+1} - H_{j+1}^{n+1}\right)\left(\frac{\zeta}{A_s^2} - \frac{1}{A_{j-1}^2} + \frac{1}{A_{j+1}^2}\right)}} \qquad \text{(A1.17)}$$

from which the coefficients in Equation A1.6 can be calculated. The inflow and outflow energy loss coefficients are calculated as:

$$\zeta_1 = \zeta_{in}\left(1 - \frac{A_s}{A_1}\right) \qquad \text{and} \qquad \text{(A1.18)}$$

$$\zeta_1 = \zeta_{out}\left(1 - \frac{A_s}{A_2}\right)^2. \qquad \text{(A1.19)}$$

A_s and ζ are determined iteratively and the description is modified to reduce oscillations when the water surface slope across the structure is small.

For undrowned flow the discharge is determined from a tabulated relationship between the discharge and the upstream water level which is calculated by MIKE 11 using a standard broad-crested weir formulation. Relationships are defined for flow through the weir in either direction. A correction factor, ε_c, can be applied to the actual discharge to allow for deviations from ideal behaviour. Figure A1.3 illustrates the calculation of the theoretical discharge from the tabulated values.

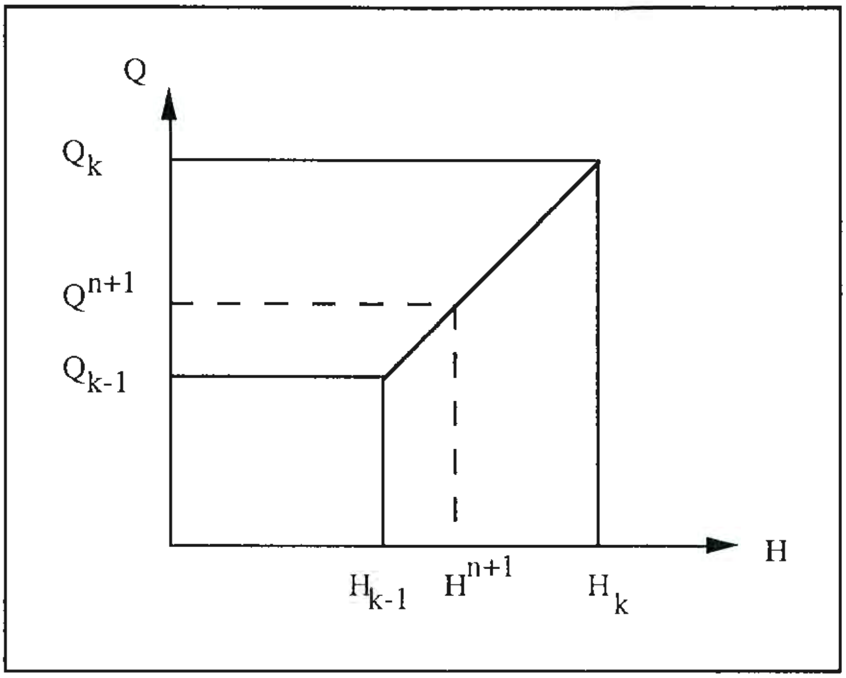

Figure A1.3: Calculation of the discharge through an undrowned weir. After DHI (1992b).

The theoretical value of Q can be calculated as:

$$Q_j^{n+1} = a_1 H_{j\text{-sign}(Q)}^{n+1} + b_1 \tag{A1.20}$$

With the correction factor applied to the theoretical discharge the coefficients in Equation A1.6 become:

$$\alpha_j = -a_1 \frac{(1 + \text{sign}(Q))}{2} \tag{A1.21}$$

$$\beta_j = \frac{1}{\varepsilon} \tag{A1.22}$$

$$\gamma_j = -a_1 \frac{(1 - \text{sign}(Q))}{2} \tag{A1.23}$$

$$\delta_j = b_1 \tag{A1.24}$$

where

$$a_1 = \text{sign}(Q) \frac{Q_k - Q_{k-1}}{H_k - H_{k-1}} \tag{A1.25}$$

$$b_1 = \text{sign}(Q) Q_{k-1} - a_1 h_{k-1} \tag{A1.26}$$

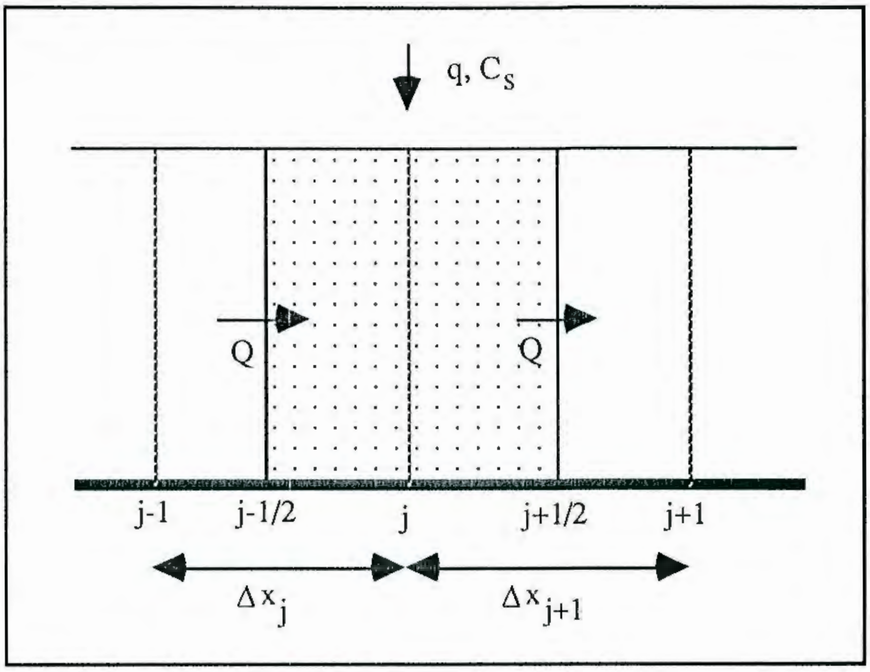

Figure A1.4: Control volume for derivation of the advection-dispersion finite difference scheme used in MIKE 11. After DHI (1992b).

A1.4 THE ADVECTION DISPERSION EQUATION.

MIKE 11 solves the following Advection-Dispersion Equation.

$$\frac{\partial AC}{\partial t} + \frac{\partial QC}{\partial x} - \frac{\partial}{\partial x}\left(AD\frac{\partial C}{\partial x}\right) = C_s q \qquad \text{(A1.27)}$$

In Equation A1.27 C is the concentration, D is the dispersion coefficient and C_s is the concentration of the lateral inflow. The finite difference scheme used to solve Equation A1.27 can be derived by considering fluxes across the boundaries of a control volume situated around grid-point j (Figure A1.4).

The continuity equation for the volume shown is:

$$\frac{V_j^{n+1} C_j^{n+1}}{\Delta t} - \frac{V_j^n C_j^n}{\Delta t} + T_{j+1/2}^{n+1/2} - T_{j-1/2}^{n+1/2} = q^{n+1/2} C_s^{n+1/2} \qquad \text{(A1.28)}$$

where V is the volume of the control volume and T is the advective-dispersive transport through the side of the control volume. The advective-dispersive transport is calculated as:

$$T_{j+1/2}^{n+1/2} = Q_{j+1/2}^{n+1/2} C_{j+1/2}^{*} - A_{j+1/2}^{n+1/2} D \frac{C_{j+1}^{n+1/2} - C_j^{n+1/2}}{\Delta x} \qquad \text{(A1.29)}$$

where $C^*_{j+1/2}$ is an upstream interpolated concentration given by:

$$C^*_{j+1/2} = \frac{1}{4}\left(C^{n+1}_{j+1} + C^{n+1}_{j} + C^{n}_{j+1} + C^{n}_{j}\right) -$$

$$\min\left\{\frac{1}{6}\left(1 + \frac{Cr^2}{2}\right), \frac{1}{4Cr}\right\}\left(C^{n}_{j+1} - 2C^{n}_{j} + C^{n}_{j-1}\right) \qquad \text{(A1.30)}$$

where $Cr = U\Delta t/\Delta x$ is the Courant number. The last term in Equation A1.30 is an explicit third order corrective term. Equations A1.28 to A1.30 can be rearranged to give Equation A1.31 which is a general implicit finite difference equation relating the concentration at three neighbouring grid-points.

$$\alpha_j C^{n+1}_{j-1} + \beta_j C^{n+1}_{j} + \gamma_j C^{n+1}_{j+1} = \delta_j \qquad \text{(A1.31)}$$

For each reach Equation A1.31 forms a series of simultaneous equations which are solved using the same iterative procedure as used in the hydrodynamic simulations. Unlike the hydrodynamic simulations, the concentration is computed at every grid-point.

APPENDIX 2

Western A.W., Hughes R.L. and O'Neill I.C., 1993. Density Stratification in the Wimmera River. Hydrology, data and modelling. Hydrology and Water Resources Symposium, Newcastle, June30 - July 2 1993, IEAust. Nat. Conf. Pub. 93/14, pp45-50.

Hydrology and Water Resources Symposium
Newcastle, June 30 - July 2 1993

DENSITY STRATIFICATION IN THE WIMMERA RIVER.

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SUMMARY Density stratification can develop in saline streams subject to low flows such as the Wimmera River. This is a significant problem because the quality of the water below the halocline is such that significant areas of the aquatic ecosystem are rendered generally uninhabitable. The physical processes involved in formation of density stratification and entrainment in a natural stream are the subject of this paper. The available literature is reviewed in the context of stratification in saline streams. Observations of entrainment and re-stratification in the Wimmera River are presented.

INTRODUCTION.

Density stratification has long been recognised in natural fluid systems such as the ocean, atmosphere, lakes, and estuaries. More recently it has been found that density stratification can develop in saline streams subject to periods of low or zero flow. One such stream is the Wimmera River located in north-west Victoria, Australia.

Density stratification has a major impact on the movement of solutes in fluid systems because vertical mixing is suppressed. This can lead to significant impacts on the water quality and aquatic ecosystem. This paper reviews the behaviour of density stratified systems in the context of the Wimmera River and presents observations of entrainment and re-stratification in the Wimmera River.

THE WIMMERA RIVER.

The Wimmera River is located in north-western Victoria (Fig. 1). The flow regime in the river becomes more variable downstream and is best described as intermittent in the lower reaches of the river. Flow events in the lower river generally occur during the period from June through to October and the flow is typically low or zero during the remainder of the year. There are also significant inflows of saline groundwater to the lower reaches of the river (approximately from Horsham to Lake Hindmarsh). During low flow periods the river contracts to a series of pools which can be as large as 10m deep, 50m wide and several Km long. These pools develop density stratification during low flow periods.

A major environmental survey of the Wimmera River was conducted by Anderson and Morison (1989). This survey identified density stratification and low dissolved oxygen levels during low flow periods as the most significant environmental water quality problems in the river. The water below the halocline in stratified pools (saline pools) is typically anoxic, contains hydrogen sulphide and has low pH. This means that the water and area of bed below the halocline is unsuitable for aerobic organisms. This is especially significant because the river bed plays an important role in the aquatic ecosystem and access to it is restricted (Anderson and Morison, 1989).

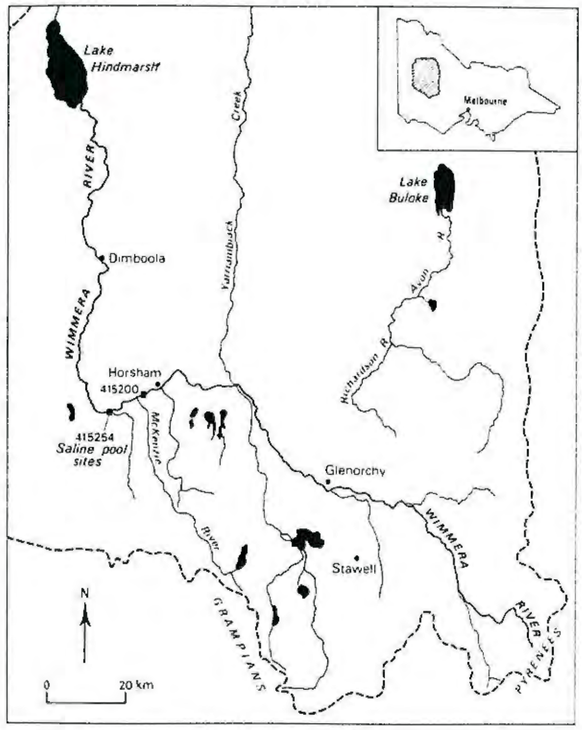

Figure 1: The Wimmera River catchment.

As part of their study Anderson and Morison (1989) in conjunction with the (then) Rural Water Commission of Victoria conducted a six week experimental release to the river during February and March 1987. A flow of 0.6 kl/s resulted in significant increase in dissolved oxygen concentrations above the halocline but little change in the position of the halocline or the water quality below the halocline. From surveys conducted after flow events of approximately 8 kl/s and 35 kl/s it was found that the smaller event resulted in mixing of shallow stratified pools and the larger event mixed most pools. Density stratification was observed to reform under low but continuous flow.

DENSITY STRATIFICATION

A number of detailed reviews of the rather extensive literature relating to stratification are available such as Fernando (1991), Pedersen (1980), Sherman et al. (1978), Thorpe (1987) and Turner (1979). These cover a wide range subject material. This paper only addresses the literature as it relates to stratification in the Wimmera River.

The major cause of density changes in the Wimmera River is variation in salinity with depth; although often there is also a difference in temperature associated with the salinity difference. The density or salinity profiles found in the Wimmera River are typically characterised by a halocline which separates two almost homogeneous layers.; although a few more complicated profiles can be found in the river.

The most important parameters characterising a density stratified system are the entrainment parameter, E = Ue/U, where Ue is the entrainment velocity and U is the mean layer velocity, or some other suitable velocity scale such as the shear velocity, and the Richardson Number, Ri. There are several versions of the Ri Number including the Gradient Ri Number (Ri_g) and the Bulk Ri Number (Ri_b).

Ri_g is the ratio of the density gradient and the square of the velocity gradient non-dimensionalised by gravitational acceleration (Turner, 1979). Ri_b is based on bulk layer properties and is the square of the inverse of the Densiometric Froude number. It may be defined as:

$$Ri_b = \frac{g'h}{U^2} \ .$$

where $g' = g\Delta\rho/\rho$ is the buoyancy, ρ is the density and h is the turbulent layer depth (Christodoulou, 1986) (Fig. 2). Ri_b and Ri_g relate shear effects and buoyancy effects and as such can be thought of as parameters reflecting the systems shear stability.

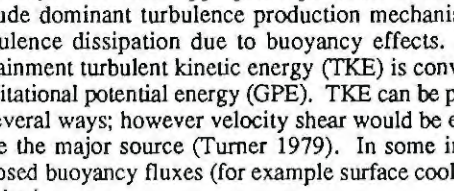

Figure 2: A two-layer stratified system.

Entrainment is fundamentally a turbulent process (Price et al. 1978) so the most appropriate parameterisation should include dominant turbulence production mechanisms and turbulence dissipation due to buoyancy effects. During entrainment turbulent kinetic energy (TKE) is converted to gravitational potential energy (GPE). TKE can be produced in several ways; however velocity shear would be expected to be the major source (Turner 1979). In some instances imposed buoyancy fluxes (for example surface cooling) can also be important.

Velocity shear occurs in the vicinity of boundaries, the interface and, when surface shear is present, at the surface (Narimousa and Fernando, 1987). TKE produced at a location remote from the interface must be transport to the interface and is subject to dissipation during that transport. Therefore TKE produced by shear at the interface would be expected to entrain fluid more efficiently than that produced remotely (Turner, 1979). This is empirically supported by observations of turbulence produced near walls which show that the turbulence largely dissipates near the wall (Hinze, 1975). The velocity scales used to define the Ri number should be based on the velocity shear at the interface. Buoyancy effects depend on the density gradient present (Atkinson and Munoz, 1988).

Since the velocity shear at the interface and the interfacial density gradient are the most significant factors controlling mixing; Ri_g is more fundamental than Ri_b. Calculation of Ri_g requires either the velocity and density gradients at the interface or some assumption regarding the thicknesses of the density and momentum interfaces. However because of the simple modelling framework proposed below Ri_b is the more convenient form of the Richardson number to use.

MODELLING STRATIFIED SYSTEMS

If density stratification is to be included in a model of a system that model must include some vertical discretization. A two layer model is the simplest that can be used. In the most complex models many layers are used and vertical diffusivities are predicted using turbulence models including including buoyancy effects.

Two-layer models have been used successfully in canals (Roelfzema, 1980), estuaries (Vreugdenhil, 1970; Hodgins, 1977), fjords (Hodgins, 1978) and the ocean (Pollard et al. 1973; Price et al., 1978; others) where they capture the overall characteristics of the system especially when the stratification tends to be characterised by sharp interfaces. These models have the advantage of being relatively simple while providing a much more realistic picture of the behaviour of a stratified system than a one-dimensional model would (Christodoulou and Connor, 1980). A two-layer model will be used to model the density stratified pools in the Wimmera River.

Within this framework individual processes must be simulated. In a two layer model vertical mixing must be predicted from bulk layer properties such as mean velocity, depth and mean density. Formation processes also need to be simulated.

FORMATION PROCESSES

Several processes leading to density stratification in rivers have been postulated. Groundwater seeping into the river could accumulate in the bottom of deeper pools as a consequence of its higher density. This postulation is supported by similarities between the conductivity and ionic characteristics of the groundwater and water below the halocline in some stratified pools (Anderson and Morison, 1989). A detailed description of this process will be developed as part of this study.

Field data collected to date indicates that stratification begins to re-establish within days of the river discharge falling to low levels. Figure 3 shows continuous conductivity records at depths of 0.5 and 2.5 meters at Lower Norton along with the discharge at Horsham (415200). The rapid increase in the conductivity at the bottom of the pool occurs when the halocline moves above 0.5m above the river bed. The flow rate when stratification is first detected was approximately 2.5 kl/s.

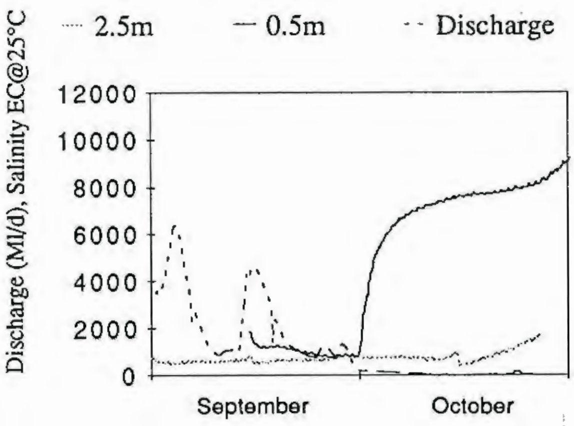

Figure 3 Formation of density stratification in the Wimmera River during September/October 1989 at 415254 as discharge decreases. Salinities are monitored at 0.5m and 2.5m from the river bed and are in EC@25°C units and discharge is at 415200 is in Ml/d.

Inflow of saline river water into the bottom of a pool as a density current could also lead to density stratification (Anderson and Morison, 1989). Another possible formation process involves a relatively fresh inflow flowing over the pool surface as a buoyant overflow. This process has been suggested by Morrissy (1979) as a mechanism for saline pool formation in the Murray River in Western Australia. We have observed this process during the beginning of a flow event in the Wimmera River. Figure 4 shows salinity profiles at Lower Norton on the 4/8/92 and the 13/8/92. On the second occasion the flow rate in the river was approximately 3.5 kl/s and the inflowing water was relatively fresh (600 EC). This water had displaced 2.5m of the original water in the pool and mixing was continuing.

Imberger and Patterson (1989) have reviewed similar processes in the context of modelling the density profile in lakes. One approach used in reservoir modelling is to add the inflowing water, with some mixing, to the layer which most closely matches its density. Models such as HEC5Q (US Army, Corps of Engineers, 1987) and DYRESM (Imberger et al., 1978) make use of this approach with apparent success. A simplified version of this approach could be used to model these processes in the Wimmera River.

MIXING PROCESSES - ENTRAINMENT

A number of mixing regimes can be identified depending on the Ri number. Very little mixing occurs at high Ri numbers. Fernando (1991) provides a useful review of behaviour at high Ri numbers including the effects of molecular diffusion. This regime will not be examined further since our interest in the Wimmera is directed more towards significant mixing events.

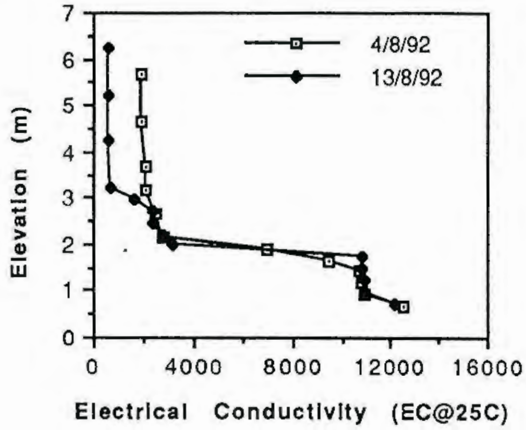

Figure 4: Formation of density stratification by buoyant overflow.

At intermediate Ri numbers entrainment is observed to be proportional to the inverse of the Ri number (Christodoulou, 1986; Fernando, 1991). This corresponds to a circumstance where the rate of increase of GPE due to mixing is proportional to the rate of production of TKE (Rouse and Dodu, 1955).

At low Ri numbers, which represent low stability, the mixing is rapid and can no longer be explained by the Ri^{-1} rule (Pedersen, 1980; Christodoulou, 1986; Fernando, 1991). The kinetic energy supplied to the fluid being entrained becomes significant in this regime. The stability of low Ri number shear flows has been examined by many authors. The classic result of this work is the Miles-Howard theorem which predicts that unbounded parallel shear flows are always stable to infinitesimal disturbances if the Ri_g is everywhere greater that 1/4 (Howard and Masloe, 1973; Miles, 1986). Other authors have also examined the problem of stability of stratified flows (Holmboe, 1962; Orlanski and Bryan, 1969; Frankignoul, 1972; Howard and Masloe, 1973; Abarbanel, 1984) and a Ri_g of approximately 1/4 has generally emerged as a stability criterion. Rapid increases in mixing occur when the flow becomes unstable.

ENTRAINMENT MODELS

There are essentially two types of simple entrainment models used. The first predicts mixing as a continuous function of the Ri number and the second assumes that mixing is negligible for Ri numbers above a critical value and that the rate of mixing is controlled by a dynamic equilibrium for Ri below the critical value.

Christodoulou (1986) has examined many experimental studies and attempted to develop a unified entrainment relation based on the Ri_b. The relation is regime dependent and is:

$$E = 0.07 \qquad Ri_b < 0.01 \qquad (1a)$$

$$E = 0.007\, Ri_b^{-1/2} \qquad 0.01 < Ri_b < 0.1 \qquad (1b)$$

$$E = 0.002\, Ri_b^{-1} \qquad 0.1 < Ri_b < 10 \qquad (1c)$$

$$E = 0.007\, Ri_b^{-3/2} \qquad Ri_b > 10 \qquad (1d)$$

It is also possible to develop entrainment relationships using the TKE budget (Hansen, 1975; Sherman et al., 1978; Spigel and Imberger, 1980; Narimousa and Fernando, 1987; Atkinson and Munoz, 1988). Equation 2 represents the TKE balance for a unit mass of fluid.

$$\frac{\partial q}{\partial t} = C_f \frac{u^3}{h} - \frac{g}{\rho}\overline{w'\rho'} \qquad (2)$$

The left hand side represents time rate of change of TKE and can be parameterised as $C_t U^2 U_e/h$. The first term on the right hand side represents the shear production and the second term the buoyancy flux which is g'Ue. C_t and C_f are parameterisation coefficients and w' and ρ' are turbulent fluctuations of vertical velocity and density respectively. Substituting these parameterisations into 2 and rearranging gives:

$$E = \frac{C_f}{C_t + Ri_b}. \qquad (3)$$

Pollard et al. (1973), Kundu (1981) and Kranenburg (1981) have all modelled wind deepening of the oceanic mixed layer by assuming a dynamic equilibrium including surface stress due to wind shear, momentum increase of the mixed layer and the momentum required to accelerate entrained fluid. It is necessary to make some assumption about vertical mixing to obtain a solution. In each case this assumption was that there was some Ri which was maintained during mixing and above which mixing was not possible. Pollard et al. assumed a critical Ri_b of O(1) while Kundu and Kranenburg both assumed a critical Ri_g of 1/4. These models all indicate that $E_* \alpha Ri_*^{-1/2}$, where * indicates that the surface shear velocity has been used as the velocity scale. The three models are equivalent in practice because the assumptions used by both Kundu and Kranenburg imply that the Ri_b is proportional to the Ri_g.

MIXING IN THE WIMMERA RIVER

A series of salinity and temperature profiles are available for a mixing event which occurred in the Wimmera River on the 15-16/8/92. The site, site 1, is located approximately 20km downstream of Horsham (Fig. 1); the maximum depth was approximately 8.5-9m and surface width was approximately 60m. The data set spans 25 hours over which time there was 2.5m of entrainment. Eighteen profiles were measured over this period. Figure 5 presents profiles measured at the start, in the middle and at the end of the period.

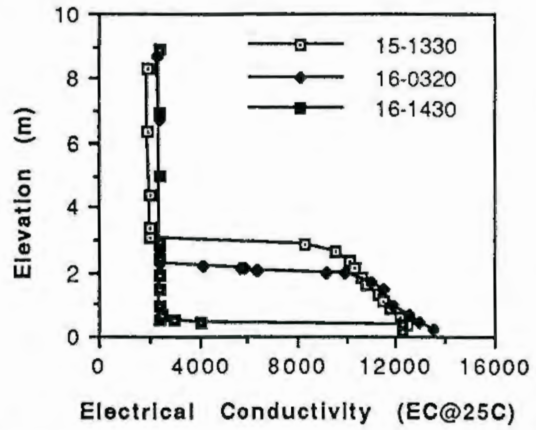

Figure 5: Salinity profiles at site 1 during a mixing event on the 15-16/8/1992.

The position of the interface can be determined by taking the mean of the two elevations which bound the region with the highest salinity gradient. It was possible to locate the salinity probe vertically to the nearest 5cm. Figure 6 shows the change in position of the interface with time. A second order polynomial was fitted to the data and can be used to calculate the mean entrainment velocity at any time. The variation in river stage is also shown.

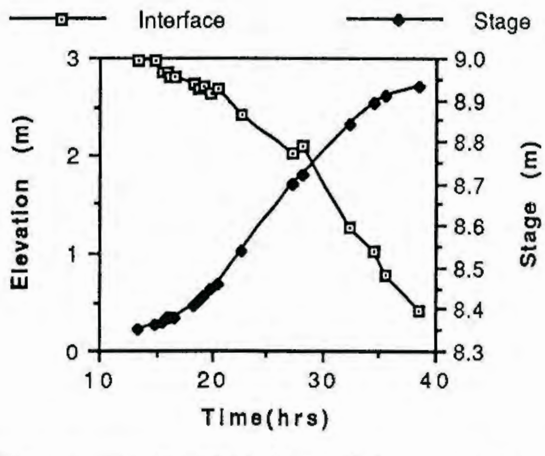

Figure 6: Location of interface and river stage at site 1 during mixing event on the 15-16/8/1992.

By dividing the discharge by the cross-sectional area above the interface it is possible to calculate the mean velocity of the mixed layer. The discharge at the site was estimated from the Horsham gauge (415200) (Fig. 1) with allowances made for tributary inflows and is subject to errors of approximately 10%. The density difference across the interface was estimated by taking the density difference over the region in which the salinity rapidly changed. Using the mean velocity, the depth of the mixed layer and the difference in density it is possible to calculate Ri_b. Figure 7 shows the variation in Ri_b with time. Error bars representing an estimated error of ±25% are shown. The entrainment parameter, which is subject to errors of approximately ±10%, is also shown.

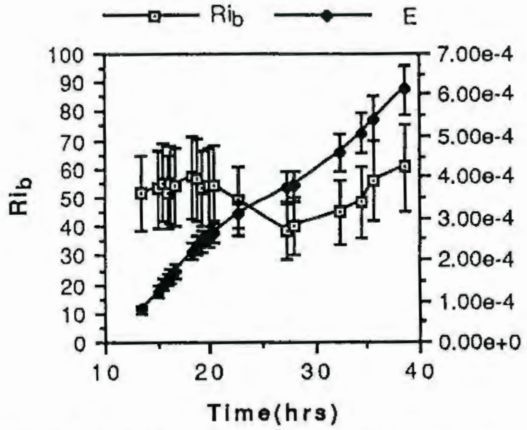

Figure 7: Variation of Ri_b and E during mixing event on 15-16/8/1992 at site 1.

Ri_b remains almost constant throughout the mixing event while the entrainment parameter increases steadily as the entrainment rate increases. There is apparently no relationship between the Ri_b and E. However the relatively constant value of Ri_b may indicate that Ri_b is being maintained at some critical value by mixing or that entrainment is being controlled by some process other than interfacial shear.

Ri_b during different mixing events can be calculated for another site, site 2, approximately 1km upstream of site 1. The data available at site 2 is a combination of manually measured salinity profiles and continuons monitoring data. The continuously monitored data consists of salinity time series measured 0.5m above the river bed. Ri_b estimated at site 2 from continuously monitored salinity are 14, 21 and 25 with an error of approximately ±30%. A Ri_b of 12 ±25% can be calculated for site 2 on the 13/8/92 (see fig. 4). These estimates of Ri_b are significantly less than those for site 1. This indicates that, if there is a critical Ri_b for entrainment from saline pools, it varies from pool to pool.

The mixing rates observed at site 1 are significantly higher than those generally reported in the literature. For example predictions of E made using Christodoulou's (1986) relationship (see previous section) are between 4 and 60 smaller than the observed value observed. Several differences exist between natural streams and the laboratory which could account for this difference.

The fact that lateral velocity profile in a river, especially on bends, is more non-uniform than for a straight rectangular section is likely to be significant. Higher velocities tend to occur in the deepest part of the section and on the outside of bends. The lower layer is also located in the deep part of the pool. A Richardson number based on the mean velocity above the stratified section of the flow would be significantly lower and thus indicate a higher mixing rate.

Other turbulence producing mechanisms may also be playing a role in entrainment. For example boundary roughness (Jones and Mulhearn, 1983; Grubert, 1989), channel irregularities (Sumar and Fischer, 1977) and secondary currents (Scranton and Lindberg, 1983; Deardorff and Yoon; 1984) can all affect entrainment presumably by influencing background turbulence levels or by enhanced interfacial shear. Breaking of internal waves at the boundaries of the lower layer could also occur thus providing another turbulence producing mechanism and adding to entrainment rates (Ivey and Nokes, 1989).

CONCLUSIONS

Density stratification has been discussed in the context of saline streams. Processes leading to the formation of density stratification in saline streams including groundwater seepage, density currents and buoyant overflows were briefly considered. Entrainment, which is fundamentally a turbulent process and needs to be examined in this context, was considered in greater detail. Observations of mixing in the Wimmera River indicate that entrainment is more rapid than that predicted by a Ri number based on the mean cross-sectional velocity and mixing rates reported in the literature. One reason for this is that the Ri_b calculated may not be equivalent to that reported in the literature due to the non-uniform lateral velocity profile. Other contributing factors related to other turbulence producing mechanisms may also exist.

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APPENDIX 3

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Note: This paper describes results of this research which are described in the body of the thesis; however this paper is not cited in the body of the thesis.

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WIMMERA RIVER: HYDROLOGY, DATA AND MODELLING

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SUMMARY The Wimmera River, located in North-Western Victoria, is a saline stream with a highly variable flow regime. A solute transport model of the Wimmera River is being developed as part of a study of the processes controlling solute transport in the Wimmera River. This paper examines data requirements for this study and methods used to determine model boundary conditions using an incomplete data set. The role of models as a tool for critical analysis and the need for and role of data are briefly examined. The calculation of temporal boundary conditions requires flow rates and concentrations for all inflows. Flows from ungauged catchments and the salinity of inflows were calculated using nearby catchments. The salinity of inflows was calculated using solute rating curves. Geometric boundary conditions are also required to calculate channel conveyance and storage. A method of stochastically infilling cross-sections was developed to provide this information. Preliminary modelling results are presented.

INTRODUCTION

Hydrologic research and water resources management are often aided by the use of numerical models. These models are used in research to study the behaviour of the system being investigated and in management to predict the likely impact of management changes. In order to perform these tasks adequately a data set that provides sufficient information about the system is required. The model is then used to interpret this data set. This paper briefly describes the hydrology of the Wimmera River, discusses data requirements for this study and presents some preliminary modelling results. The discussion is conducted in the context of a model based on deterministic descriptions of the physical processes occurring in the river.

THE WIMMERA RIVER

The Wimmera River is located in north-west Victoria, Australia. It rises in the Pyrenees Ranges and flows generally north-west collecting runoff from tributaries in the Grampians ranges and the foothills of both the Pyrenees and Grampians. Yarriambiack Creek is a distributory stream which leaves the Wimmera upstream of Horsham. Downstream of Horsham the river turns north and flows to Lake Hindmarsh where it usually terminates (Fig 1.). In wet periods it sometimes continues further into a system of terminal lakes. Nearly all the surface runoff entering the river is generated in the ranges and their foothills. No significant tributaries enter the river after it turns north.

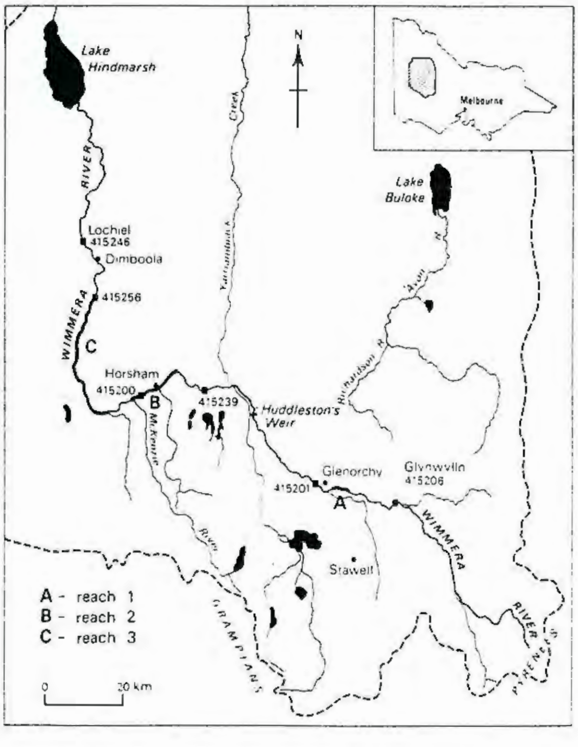

Figure 1: The Wimmera River Catchment

Flow Regime

The flow regime in the Wimmera River becomes increasingly variable downstream. Table 1 presents the mean annual flow and coefficient of variation at several gauging stations along the river. Flows are typically low and can be zero during the period from November to May inclusive while during the period from June to September a series of flow events would be expected. October is

typically a month of transition from high winter/spring flows to low summer flows. Like the annual flows in the Wimmera River the seasonal pattern of flow is also variable.
The Wimmera River is an important source of water for the Wimmera Mallee Stock and Domestic Water Supply (WMSDS) system. Flows from the river itself are harvested at Glenorchy and Huddleston's weir and several storages exist on tributaries of the river. These diversions represent 48% of the 209,000 Ml annual yield of the catchment (Hooke, 1991). Diversions from the Wimmera River have the greatest impact on low flows since the storages are off-stream and the diversions have limited capacity. At present there are no environmental flow provisions for the river and diversions are only limited by channel and storage capacity.

Site	$\overline{Q}$ (Ml)	C.V.	Flow Weighted mean EC	median EC
415206	67700	0.88	430	2400
415201	92700	0.92	420	2100
415200	139000	1.17	300	1000

Table 1: Flow and salinity characteristics at various locations on the Wimmera River.

Water Quality Regime

High salinity levels in the Wimmera River are a matter of concern from both the water supply and environmental perspectives. Flow-weighted mean salinities and median salinities are presented in Table 1 following Hooke (1991). There is an inverse correlation between flow and salinity and therefore the seasonal pattern of salinity is opposite to the seasonal flow pattern. Significant salt loads are generated in both the upper and lower catchment. The salt loads entering the river in the lower catchment are due to significant inflows of saline groundwater.

A major study of the aquatic environment in the Wimmera River was undertaken by Anderson and Morison (1989). This identified density stratification and low dissolved oxygen levels as the major environmental water quality problems in the river. The density stratification is due to differences in salinity over the water column which usually result from saline groundwater entering the river (see Western et al., this conference). Low dissolved oxygen (DO) levels are associated with the stratification and with low flows during summer. An experimental environmental release of 50 Ml/d was conducted as part of Anderson and Morison's study resulting in significantly improved DO levels above the halocline but no change below the halocline.

The aim of this project is to develop an understanding of the processes involved in solute transport in the Wimmera River, especially those associated with density stratification. Part of the methodology being used involves development of a model of solute transport in the river. A data collection program designed to provide the information necessary to develop and test this model is also being conducted. The knowledge gained in this process will then be used to examine the impact of various management options on the salinity of the river and on diverted salt loads.

MODELS AND THEIR PROBLEMS

Why model? Models provide a tool for critical analysis (Konikow and Bredehoeft, 1992). Models provide a theoretical basis for organising and interpreting data (Hillel, 1986; Grayson et al. 1992). They allow the testing of hypotheses (Konikow and Bredehoeft, 1992; Grayson et al. 1992) and the process of model development can lead to new insights into the behaviour of systems, the identification of sensitive parameters and indicate possible further investigations (Hillel, 1986; Konikow and Bredehoeft, 1992; Grayson et al. 1992).

Hypothesis testing is a fundamental element of scientific method. It is only by advancing and testing hypotheses that knowledge advances. Indeed many would argue along the lines of Popper (1959) that scientists can never validate a hypothesis and therefore we only increase our knowledge when we falsify a hypothesis (Konikow and Bredehoeft, 1992). Any test of a hypothesis should be conducted so that clear evidence of the suitability or otherwise of the hypothesis is obtained. Therefore any test of a hypothesis, including the hypothesis that the model which we advance is a good description of the system being modelled, must be capable of falsifying that hypothesis (Klemes, 1982, 1986; Dooge, 1986).

A problem that plagues hydrologic modelling generally is that many problems are poorly posed and model parameters are not unique (Philip, 1975; Klemes, 1982; Beven, 1989; Konikow and Bredehoeft, 1992; Grayson et al., 1992). Observations of system output are integral quantities resulting from a large amount of averaging. These can be reproduced by a model using different parameters and representations of variability. Therefore tests based on system output lack the power (in the statistical sense) to distinguish between model set-ups and between hypothesized representations of reality. Similar issues arise in river modelling which is conducted in the context of boundary conditions subject to error; channel variability in both space and time; parameters that vary in space, with discharge and over time and imperfect process descriptions. This implies that more detailed data that directly reflects the impact of specific processes need to be collected in many modelling studies (Dunne, 1983; Klemes, 1986; Grayson et al., 1982).

WHAT PURPOSE DO DATA SERVE?

Model development and evaluation are carried out by comparing model predictions with the historical behaviour of the system. Improvements are made in an iterative process of parameter adjustment or calibration and possibly, changes to the conceptual framework of the model.

Data provide information required to develop and evaluate a site specific model. They define external influences acting upon a system such as inflows to a river reach and the response of the system, such as the routing of a flood wave, which is a function of the processes operating. During modelling, data defining external influences are used to specify boundary conditions and the observed system response is used to develop/calibrate and test a hypothesized representation (the model) of the system. The extent to which either of these purposes is fulfilled depends fundamentally on the appropriateness and quality of the data.

Boundary condition data must define the external influences on spatial and temporal scales which are appropriate to the scales at which the modelling is being conducted (Cunge et al., 1980). Ideally the influence should be directly measured; however it is possible to use a surrogate from which the external influence can be defined.

In the case of system response, the quantity for which data are collected must be sensitive to the process being studied. As discussed above the system output is often insensitive to internal processes. Therefore the internal response of the system needs to be observed when evaluation of internal processes is necessary. Although necessary, data collection is expensive and priorities need to be set. While it is not always possible to collect an ideal data set; identification of the ideal data set and its purposes will enable better priorities to be set. These priorities need to be reassessed as data is collected and analysed.

The type of modelling conducted and the way a model is set up should reflect the amount of data already available or that can be collected as part of a special data collection program. The less detailed the available data then the simpler the modelling should be and the more a priori reasoning needs to be used. A review of the data requirements, methods used to obtain boundary conditions and a data collection program designed to provide additional data for a study of solute transport processes in the Wimmera River is presented below.

MODEL DATA REQUIREMENTS

The model of the Wimmera River being developed is based on a one-dimensional link-node river model. The MIKE11 (DHI, 1988) software package is being used. This model simulates flow using the St Venant equations or simplifications there of and solute transport using the advection-dispersion equation. Modifications will be incorporated in the solute transport component of this model to enable a two-layer description of stratified pools.

The model incorporates the main stem of the Wimmera River from Glynwylln to Lochiel which is approximately 190km by river. Tributary inflows, diversions, inflows of groundwater, the advection and dispersion of solute and processes associated with the stratification in pools will be simulated.

This model requires data to define temporal and geometric boundary conditions and data to develop/calibrate and test the model. Calibration data will be briefly discussed then boundary condition data will be discussed in greater detail.

Development, Calibration and Testing

The features being simulated are the routing of water down the river system including the flow depths at different flow rates; the passage of solute down the system and the behaviour of stratified pools under various flow conditions. Flow predictions will be used as input to transport simulations.

Stage , discharge and salinity hydrographs are required to calibrate and test the flow routing and advection-dispersion components of the model. Gauging sites are required at intervals short enough to define the routing of water and solute down the system. Five stage recorders are located along the Wimmera River downstream of Glynwylln, four of which are rated. These sites will be used to calibrate the flow routing component of the model. Data loggers measuring electrical conductivity have been installed at gauging stations along the river to provide the required salinity data.

Since stratified pools are essentially a local phenomena, the information required to study the processes associated with stratified pools is totally different to that required for the above calibration purposes. The feature that characterises the state of these pools is the variation of salinity over the water column and data characterising this variation is being collected using several different methods.

BOUNDARY CONDITION DATA

Temporal boundary condition data consisting of time series of discharge and salinity at each inflow and the stage discharge relationships or time series of discharge at each outflow are required. Geometric boundary conditions are also required. These consist of a series of cross-sections for each reach, hydraulic structure information and channel connectivity (Cunge et al., 1980).

Temporal Boundary conditions

The available set of flow and salinity data required to define temporal boundary conditions is incomplete. Therefore some flows and salinities need to be estimated from surrogate variables.

The following techniques were used to estimate ungauged flows with preference given to methods in the order of their discussion. If gauged flows were available for part of the period then a correlation with a similar stream was adopted. When no flows were available the hydrograph from a similar stream was scaled by the ratio of the mean annual discharges. The mean annual discharge was estimated by a water balance if sufficient gauging information for the receiving stream was available or by the following relationship between catchment area, A, and mean annual discharge, Q for similar streams.

$$\frac{Q_1}{Q_2} = \left(\frac{A_1}{A_2}\right)^{0.8} \qquad (1)$$

The index in equation 1 for the Wimmera and neighbouring catchments was found to be 0.8 for unregulated streams.

Infilling Salinity Data

For streams in the Wimmera most existing salinity data were collected when rating measurements were conducted and, in some cases, on a regular monthly basis. A continuous salinity monitoring program has been initiated to monitor the major salt loads entering the river. Continuous monitoring along the river will also allow an assessment of the salt loads entering the river in groundwater inflows which are a major source of salt load.

Methods used to infill concentration data need to be selected in the context of processes controlling water quality. Walling (1984) provides a review of many of these. The most important process is normally the dilution process in which solute rich baseflow is diluted by solute poor surface runoff. Spatial variability in runoff generation also influences the source and thus the concentration of runoff. The concentration of solute in flow from a particular source also varies on seasonal and event time scales. Since the catchment discharge integrates the flow determining processes while solute concentration integrates both the flow determining processes and the processes determining the solute concentration of each flow source, solute concentration will be more variable than discharge.

There are several methods for infilling solute concentration data that have been collected infrequently. Regression models incorporating discharge and other variable that reflect seasonal variation, antecedent conditions or flow source can be developed. Alternatively more complex models that account explicitly for the solute generation processes could be used; however these models are not considered appropriate due to their complexity and large data requirements. Several regression models were assessed for the Glynwylln gauging station where the single largest solute load of all the unregulated flows to the river model occurs.

The monthly salinity data set used for this assessment covers the period September, 1975 to June, 1988. This data set was divided into two by extracting data for every second month. A log-linear relationship between flow, Q, and salinity, C, was used as the basis for each model (Eq.2).

$$C = a\,Q^{b} \tag{2}$$

In addition to this basic model the following models were assessed. A minimum variance unbiased estimator for log-linear regression (Cohn et al., 1989) (MVUE model); a model in which a is determined by linear interpolation from the two neighbouring samples (Walling, 1984); a model using baseflow, Q_B, rather than total flow, Q, as the explanatory variable; a model using total flow and the ratio of total flow to baseflow, $B=Q_B/Q$, and a model incorporating the 45 day antecedent flow, Q_{45}. Table 2 presents performance measures that indicate the goodness of fit of the original model, the absolute errors for the predictions of conductivity and load and the sum of squared prediction errors. The coefficient of efficiency (Aitken, 1973) for concentration and load estimates is also provided. Except for R^2, these performance measures are presented in the real domain since this is the domain in which the predictions are to be used (McCuen et al., 1990).

Of the three models based on the log-linear rating curve the MVUE provides the most accurate predictions of conductivity and load; however the difference between the log-linear rating curve and the MVUE is not large. The variability of the predictions of the log-linear rating curve and MVUE are similar and significantly less than Walling's interpolation model. It is interesting to note that the inclusion of additional, statistically significant, variables does not necessarily lead to an improvement in prediction and that the accuracy of concentration prediction is not a good indicator of the accuracy of load predictions.

Inflows from the WMSDS are treated differently as they originate in storages which would act as buffers thus removing short term variation associated with discharge. Conductivity data for these flows can be infilled using linear interpolation between samples.

For ungauged tributaries it is not possible to directly determine a solute rating curve. In these cases the slope of the solute rating curve will be assumed to be the same as that for similar catchments and the coefficient will be calculated so that the annual salt load is predicted correctly. The annual salt load is obtained either from mass balance techniques if sufficient information is available or from an assumed salt load per unit area.

This technique is based on intuitive reasoning and some empirical evidence rather than detailed study. It can be argued that the index, b, of the rating curve results from systematic variation in the strength of runoff sources and

Method	Model	R^{2a}	Cumulative Error		Sum of Squared Errors	
			Conductivity	Load	Conductivity	Load
Log-Linear	$EC = 6210\,Q^{-0.31}$	0.70	14.3 E 3, 7.4%[b]	335 E 3, 4.6%	77.9 E 6, 61%[c]	290 E 9, 92%
MVUE	$EC = 6210\,Q^{-0.31}$, bias adjusted	N.A.	-3.49 E 3, -1.8%	-255 E 3, -3.5%	81.7 E 6, 59%	251 E 9, 93%
Walling	$EC = a\,Q^{-0.31}$, a interpolated	N.A.	-8.50 E 3, -4.4%	-300 E 3, -4.1%	242 E 6, -21%	464 E 9, 87%
Baseflow	$EC = 6300\,Q_B^{-0.33}$	0.62	12.0 E 3, 6.2%	-1727 E 3, -23.9%	71.0 E 6, 64%	154 E 9, 96%
Baseflow index	$EC = 13100\,Q^{-0.40} * e^{-0.724B}$	0.63	9.12 E3, 4.7%	-1039 E 3, -14.4%	71.2 E 6, 64%	390 E 9, 89%
Anteced' flow	$EC = 17100\,Q^{-0.14} * Q_{45}^{-0.21}$	0.68	-1.50 E3, -7.8%	-1330 E 3, -18.4%	43.1 E 6, 78%	502 E9, 86%

[a] Coefficient of determination   [b] Percentage cumulative error   [c] Coefficient of efficiency

Table 2. Comparison of errors associated with different solute rating models for the Wimmera River at Glynwylln

that hydrologically similar catchments should, at least to a first approximation have similar indices for their solute rating curves. Empirical support for this assumption comes from seven unregulated subcatchments of the Wimmera catchment six of which have values of b which are statistically similar to 0.3. The data for the remaining catchment exhibits a large amount of scatter and no statistically significant relationship between salinity and flow. It must be emphasised that this is simply a means of calculating salinities that should provide a qualitatively realistic temporal variation and nothing more.

Geometric Boundary Conditions

The geometric boundary conditions required for the model include cross-sections, hydraulic structures and channel connectivity. Information defining the latter two is generally relatively easily obtained from maps and plans and these will not be considered in this discussion. On the other hand, a set of cross-sections that defines the variation in the shape of the stream channel is often not available. This section deals with a method for stochastically infilling these data.

Natural stream channels are variable on both the catchment scale and the local scale (Richards, 1982). On the catchment scale there is generally a tendency for the river to become wider and deeper and for the width to depth ratio to increase downstream. The channel slope also tends to decrease downstream (Richards, 1982). On a local scale the channel alternates between pools and riffles and this introduces irregularities in the cross-sectional area and bed slope which occur on a scale similar to the pool riffle spacing which is generally approximately 5-7 times the mean channel width.

The effect of these irregularities is to introduce irregular pressure and gravity forces in the flow direction (Chiu and Lee, 1971). The volume of water stored in the channel must also be increased due to the backwater effects generated by hydraulic controls in the channel. These effects are most pronounced at low flows (Miller and Wenzel, 1985).

These characteristics suggest that cross-sections could be generated stochastically if the variation of the river channel can be characterised statistically. Chiu and Lee (1971) used a method based on perturbing an assumed mean cross-section to do this; however only the hydraulic characteristics of a cross-section (top width, area and hydraulic radius) are required for a one-dimensional model and a method of generating these characteristics directly has been developed for the Wimmera River for in-bank flows. The development of this methodology is described briefly.

Field observations suggest that the Wimmera River channel alternates between areas where there is an anabranching pattern, areas with a series of pools and riffles and areas with large deep pools. Initially the river was divided into three categories, denoted as wetland, channel and large pool respectively, based on these differences. The channel category was divided into channel-upstream (reach 1, Fig. 1) and channel-downstream (channel category within reaches 2 and 3, fig. 1) since the cross-sections in this category come from two distinct reaches of river.

The cross-sections available were then characterised as follows. Power curves relating top width, W, area, A, and hydraulic radius, R, to depth, D, were fitted (eq 3, 4, 5).

$$W = a\,D^{b} \quad (3)$$
$$A = c\,D^{e} \quad (4)$$
$$R = f\,D^{g} \quad (5)$$

A linear regression was fitted to the inverts of the available cross-sections over reaches of channel for which the mean slope could be assumed constant and the residuals, r, were taken as a measure of vertical position. A similar regression was used to define the top of bank. This provided seven variables characterising each cross-section. These variables were then examined to establish interrelationships. It was found that c could be predicted from a ($R^2 = 0.96$) and that e could be related to b ($R^2 = 0.73$). Both f and g have low variability (C.V. = 9% and 6% respectively) and they are therefore treated as constants. Using these relationships the number of primary variables characterising each cross-section is reduced to three.
An examination of the statistical differences between the three categories and between channel-upstream and channel-downstream was undertaken using Hotelling's T^2 test (Harris, 1985). This indicated that there was a statistical difference (5% sig. level) between the wetland category and other categories and between channel-upstream and channel-downstream. However there was not a significant difference between the channel and large pool categories which were subsequently combined and denoted as channel.

The longitudinal characteristics of a, b and r were then examined by dividing the river into three reaches (fig1). The channel tends to become shallower downstream and longitudinal trends in the means of a, μa; and b, μb; and in the standard deviation of r normalised by the mean width, $\sigma r/\overline{W}$; indicate that the width to depth ratio, $\overline{W}/\overline{D}$, is increasing, that the channel invert is becoming more variable and that the channel shape is changing from more triangular to more U shaped since b is decreasing. These trends are indicated in table 3 which provides the above variables for the channel category for three locations along the river.

The distributions of a, b and r were examined using a chi-

Locality	μ_a	μ_b	μ_c	μ_e	σ_r	$\overline{D}$	$\overline{W}$	$\overline{A}$	$\sigma_r/\overline{W}$	$\overline{W}/\overline{D}$
Reach 1	6.04	0.85	3.54	1.78	0.22	9.36	40.4	190	0.005	4.32
Reach 2	14.04	0.77	8.53	1.68	0.94	5.40	51.4	145	0.018	9.53
Reach 3	18.43	0.55	11.88	1.55	1.13	5.87	48.8	185	0.023	8.31

Table 3: Variation in cross-sectional characteristics along the Wimmera River.

squared test and it was found that these were not statistically different from normal for b and r and from lognormal for a. The data set was examined for cross-correlation which was found to be significant (t-test 5%)between log(a) and b but not between log(a) and r or b and r. Serial correlation for log(a), b and r was also examined and was insignificant (t-test 5%) for the categorised data.

This analysis was then used as the basis for a stochastic model of the cross-sections of the Wimmera River. The means of log(a) and r were calculated as a function of chainage for each category and the mean of b was calculated from the mean of a. It was assumed that the coefficient of variation for each variable was constant and sets of the three variables were then generated from normal distributions. Finally c and e were calculated using deterministic relationships with a and b.

PRELIMINARY MODELLING.

A model of the Wimmera River is being developed at the time of writing this paper. Because of the methods used to obtain data required for modelling the system the model being established is being kept as simple as possible and as much a priori reasoning is being used as possible. For example it is planned that the model of saline pools will be entirely based on a priori reasoning and external estimates of groundwater inflows. Significantly more intensive monitoring of salinities is also being conducted as part of the project.

Some preliminary results based on simulations for the period May to October 1989 for flow and 20/8/1989 to 28/9/1989 for salinity are presented in figures 2, and 3. Gauging station 415256 is used because it has both continuous flow and salinity data available. Results are similar at other stations. Boundary conditions calculated using the methods described above were used and constant model parameters have been used throughout the model. Neither density stratification nor groundwater inflows are included in the model.

The results of flow routing simulations show a tendency for simulated events to be routed down the system too quickly. This is possibly due to the effect of backwaters, which are common on the Wimmera, being ignored at this stage. Low flows are also poorly simulated. This is a result of the method used by MIKE 11 to overcome the small depth problem and this method will be altered. Simulations of stage are inaccurate at present and are sensitive to the stochastically generated cross-sections we have used; however the amplitude of stage fluctuations is generally correct. Stage in a river is determined by controlling sections as well as the imposed discharge and average channel dimensions. Therefore simulated stage is likely to be sensitive to the generated cross-sections.

Salinities were only simulated for the shorter period because problems with flow simulations at low flows meant that meaningful salinities could not be calculated. The results of the salinity simulations indicate that salinities may be slightly underestimated on average and that fluctuations in salinity are overestimated. Neglecting groundwater inflows which represent a large portion of the annual salt input to the lower section of the river means that salinities should be underestimated although the effect during this high flow period would less than at lower flows. It is also possible that inclusion of groundwater inflows would modulate salinity fluctuations since the salt entering the river during low flows would be mixed and transported during high flows; however further work is required to confirm this.

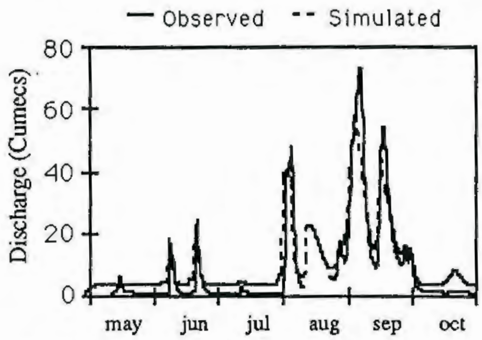

Figure 2: Simulated and observed discharge for the Wimmera River at Upstream of Dimboola (415256)

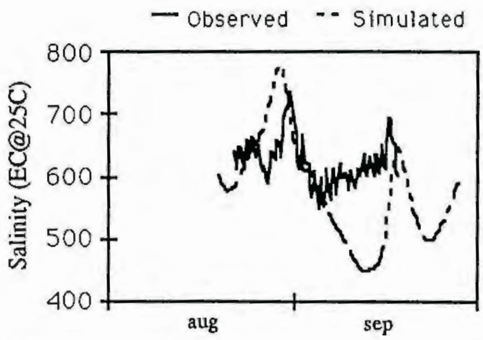

Figure 3: Simulated and observed salinity for the Wimmera River at Upstream of Dimboola (415256)

CONCLUSION

Modelling plays an important role in hydrologic studies by providing an analytic tool. However the results of modelling must be considered in the perspective of the level of understanding of the hydrologic behaviour of a system and the amount of data available. While it is not possible to use a model to improve on the amount of available data it is possible to utilize the available data in the most efficient manner possible. To do so requires that the hydrologist spend the time to consider how data may be best utilized and also requires that hydrologists be willing to use their intuition and understanding of the basic hydrologic processes when utilizing the available data. A data collection program focussed on the needs of the particular study is also of invaluable assistance and such programs should be considered for all studies. The complexity of any modelling used (and many models can be set up at varying levels of complexity or simplicity) needs to be considered in conjunction with data availability and the reporting of model results should explicitly acknowledge the uncertainty associated with them.

Several methods have been used to infill boundary condition data for a flow and salinity model of the Wimmera River. The methods used are purely operational in that they are simply used as a means of obtaining boundary conditions. However the methods used have been based on our present understanding of hydrologic behaviour and existing data available in the catchment. Such methods must be based on an understanding of the catchment being studied and must be developed from a priori reasoning. Models making use of data estimated in this manner must remain simple, in keeping with the quality of data available. Critical areas can be identified during the modelling process thereby allowing more efficient use of resources for additional data collection. While the use of directly measured boundary conditions would be superior it is felt that the methods used capture the general behaviour of the system.

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APPENDIX 4

Western A.W., Nolan J.B., Hughes R.L., O'Neill I.C. and McMahon T.A., 1993. Density Stratification in the Saline Streams of South Eastern Australia. Eos, Trans. Am Geophys. Union, 74(43): pp311-312.

Note: This abstract describes results of this research which are described in the body of the thesis; however this paper is not cited in the body of the thesis.

Density Stratification in the Saline Streams of South Eastern Australia.

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Stream salinisation associated with rising water tables is a significant problem in South Eastern Australia. Salinity induced stratification is observed in some of these streams. Flow regimes in these streams are characterised by extended periods of low to zero flow from November to June and a series of rainfall induced flow events during the remainder of the year.

The Wimmera River in North West Victoria, Australia, is an example of a density stratified stream and is the subject of a current solute transport study. The processes associated with density stratification are being studied in the laboratory and the field. A hydrodynamic model of the stream which incorporates the effects of density stratification is also being developed.

During low flows the streams essentially become a series of pools which may be totally isolated from each other or linked by shallow sections of channel. The stratification in these pools develops predominantly as a result of saline groundwater entering the river and collecting in scour holes. Buoyant flows entering the pools from upstream and either flowing over the surface of the pool and leaving denser fluid in the deepest area or along the bed of the pool and collecting in the deepest area can also lead to density stratification.

At higher flows the denser water is mixed into the overflowing stream and the stratification is destroyed. Field data collected indicates that scouring of the interface by turbulent eddies, which is a common mixing mechanism in stratified flows, can account for approximately 10% of the observed mixing. It is believed that a mechanism involving a continuous flow out the downstream end of the pool and subsequent turbulent mixing explains the observed mixing.

APPENDIX 5

Western A.W., O'Neill I.C. and McMahon T.A., 1994. Sophisticated Stream Modelling: Some Practical Limitations. Hydrology and Water Resources Symposium, Adelaide, November 21 - 25, 1994. In press.

Note: This paper describes results of this research which are described in the body of the thesis; however this paper is not cited in the body of the thesis.

Sophisticated Stream Modelling: Some Practical Limitations.

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Summary: A model of flow and salinity in the Wimmera River has been developed using stream modelling software whic solves the St Venant and Advection-Dispersion Equations. A number of issues relating to modelling methodology and technolog which have arisen are discussed. These include problems related to the limited availability of data which has led to difficulties i model calibration and testing and to the conceptualisation of a number of important components of the system. As a result th physical basis of the model has been compromised. Issues relating to scale, the benefits of modelling and generalised models ar discussed briefly. Finally it is argued that a simpler model could have been more appropriate for this study and that model choice i still a critical step in any modelling exercise.

INTRODUCTION.

The Wimmera River is a highly saline, perennial-ephemeral stream located in North Western Victoria. A physically based, hydrodynamic model has been applied to a 190 km section of the stream as part of a stream salinity study. This model is best seen as a regional scale model since it applies to more than 50% of the length of the main stem of the Wimmera River. The model was to serve as a research tool for developing an understanding of the solute transport processes operating in the stream and then, at the completion of the study, as a management tool for assessing the impact of different flow management strategies on river salinities. Since stream salinity was the major interest in this study, simulation of over bank flows was not required. Drawing on this application, the suitability of models of this genre for large scale modelling of water quality in streams is discussed.

THE WIMMERA RIVER.

The flow regime of the Wimmera River becomes increasingly variable downstream. The mean annual discharge at Horsham is 139 000 Ml and the coefficient of variability of annual discharge is 1.17. While the seasonal flow regime is also variable, it is typified by low or zero flows during the period from November to May with a series of flow events expected between June and October. During low flow periods the stream contracts to a series of long (up to ~ 3 km in length) pools which may be linked by shallow flowing sections of channel.

Diversions from the Wimmera River to the Wimmera Mallee Water Supply System (WMWSS) occur at Glenorchy Weir and at Huddlestons Weir. The diversion at Glenorchy supplies water directly to the distribution system while that at Huddlestons Weir diverts water to off-stream storages. These diversions account for 48% of the annual yield of the Wimmera River Catchment (Hooke, 1991).

High salinity levels in the Wimmera River are a matter of concern from both the water supply and environmental perspectives. The flow weighted mean salinity at Horsham is 300 μs/cm and the median salinity is 1000 μs/cm (Hooke, 1991). Significant salt loads are generated in both the upper

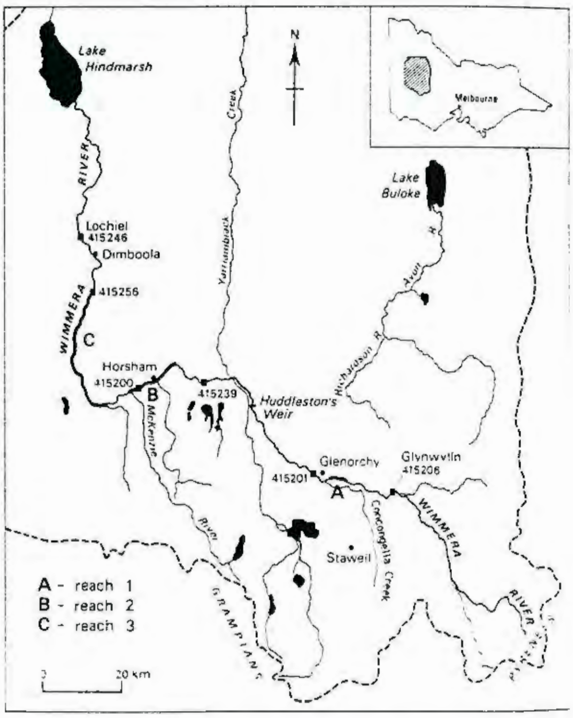

Figure 1: The Wimmera River Catchment

and lower parts of the catchment. Those in the low catchment are the result of direct intrusion of groundwater int the stream channel.

During low flow periods intrusion of groundwater into th river downstream of Horsham results in the formation c density stratification in some pools (Anderson and Morisor 1989; Western et al., 1993a). This stratification is typicall located within scour holes and is of environmental concer because of extremely poor water quality below the haloclin (Anderson and Morison, 1989). The solute transpo processes associated with these stratified pools will b included in an extension of the current river model.

THE HYDRODYNAMIC MODEL.

The modelling software nsed in this study was MIKE 11 (DHI, 1992). The hydrodynamic component of MIKE 11 is a one-dimensional link-node model which solves the St Venant Equations in an irregular channel using the Abbott and Ionescu (1967) finite difference scheme. Algorithms to model a number of hydranlic structures, including broad-crested weirs, are also incorporated in MIKE 11. Solute transport is simulated using the one-dimensional advection dispersion equation which is solved using a third order correct finite difference scheme (DHI, 1992). MIKE 11 is an excellent model typical of state of the art one-dimensional computational hydraulic models. While a specific software package has been nsed in this study; comments made abont the appropriateness of this genre of models for stndies of this type apply to all such models.

DATA AVAILABILITY.

The application of a model like MIKE 11 to the prediction of salinity requires several types of data. These include cross-sections, continuous discharge and salinity data for any inflows and either continuous discharge data or an appropriate stage-discharge relationship for any outflows. It may be appropriate to substitute stage data for discharge data in some circumstances; for example at the seaward boundary of an estuary. To calibrate and test the model, continuous salinity and gauging data are required at points within the model domain and at the model outflow.

One of the problems encountered when modelling the Wimmera River was the shortage of available data. This meant that a significant amount of infilling was required to develop model data sets. The available data and methods used for infilling that data are discussed briefly below. More detail is available in Western et al. (1993b).

Continuous discharge and salinity data were required for all inflows of surface water and groundwater to the Wimmera River. Continuously monitored discharge data were available and used as the model boundary condition at Glynwylln which is the largest inflow to the model. Tributaries and channels were represented as lateral inflows. Daily inflows to and outflows from the river associated with the WMWSS at Glenorchy were available (Fig. 2). All tributaries had at least some ungauged area. Mean daily flows from ungauged catchment areas were estimated by scaling discharge data from Concongella Creek and added to any gauged flows to obtain the lateral inflows.

Salinity data were also required for all inflows. At the start of this project, only limited spot salinity data were available at gauging stations. Although continuous salinity recorders were installed at a number of sites in 1992 and 1993, it has been necessary to estimate the salinity of almost all inflows. At Glynwylln this was achieved by using a regression relationship between flow and salinity. At other locations the relationship between flow, Q, and salinity, S, was assumed to be a power curve of the form:

$$S = a\,Q^{-0.3} \qquad (1)$$

The index in this relationship was set at -0.3 on the basis of observed relationships for several unregulated streams in the Wimmera Catchment (Western et al., 1993b) and parameter a was calculated to obtain an appropriate annual salt load. This salt load was calculated from the median salt load per unit area (Hooke, 1991) for unregulated, gauged catchments within the Wimmera Catchment.

Groundwater inflows were incorporated in the salinity model (but not the flow model) because they represent a significant source of salt. Constant flows were assumed and salinities were set equal to the local groundwater salinity. Groundwater inflows were estimated from flow net calculations (D.Strudwick, per. com., 1993) for the river downstream of Roseneath (gauge 415239) and from low flow water balance for the river between Glynwylln and Glenorchy. Groundwater potentiometry indicates that between Glenorchy and Roseneath any flow must be from the river to the groundwater (C. McAuley, per. com., 1993). No groundwater interaction was included in this central section since the impact on river flow and salt load is relatively small.

Cross-sections were required for all modelled reaches but were only available for some reaches (Reaches 1, 2 and 3, Figure 1) of the Wimmera River. The available cross-section data were extrapolated using a stochastic cross-section model. The shape of the cross-section was represented with a power curve relating width, W, and depth, D.

$$W = a\,D^{b} \qquad (2)$$

a and b were obtained by fitting this relationship to the available cross-sections. The vertical displacement of cross-sections from the mean channel grade line, r, was also calculated. The statistical characteristics of a, b and r were examined and used to infill cross-section data. This analysis accounted for channel reaches with distinctly different characteristics by categorising the river into areas with and without significant anabranches (Western et al., 1993b). Areas with significant anabranching are loosely termed wetlands. Hydraulic parameters including top width, cross-sectional area and hydraulic radius were then calculated from the generated cross-section geometry.

THE MODEL NETWORK.

The model network used to model the Wimmera River is shown in Figure 2. The model concentrates on the main stem of the river between Glynwylln and Lochiel (Fig. 1). Since simulation of over bank flows was not required, no attempt was made to include the flood plain in the model.

Channels were modelled using the diffusive wave approximation of the St Venant Equations. A number of weirs exist along the river which were modelled as broad crested weirs.

Inflows to the model occur at Glynwylln and as lateral inflows at locations marked on Figure 2. Groundwater inflows occur between Glynwylln and Glenorchy and between Roseneath and Lochiel. The two locations where significant interaction between the WMWSS and the River occur are Glenorchy Weir and Huddlestons Weir. Outflows from the model occur at these two locations, at Yarriambiack Creek and at Lochiel.

The distribution of water at all bifurcations was modelled using either a timeseries (Glenorchy outflow) or purpose written subroutines. The only stage-discharge relationship of significance in the model is that specified at Lochiel which was obtained from the gauging station rating curve.

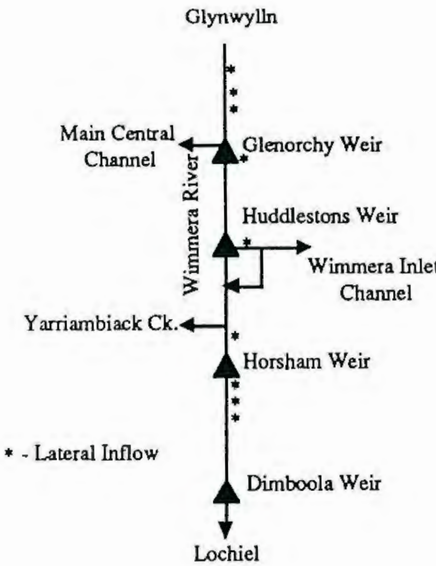

Figure 2: The model network.

Special Features.

Three features of the Wimmera River System were difficult to incorporate in the model. Firstly the river and water supply system near Huddlestons Weir is very complicated and there is little gauging data available in this area. At Huddlestons Weir water is diverted to offstream storages via the Wimmera Inlet Channel. A number of ungauged tributaries enter this channel downstream of the river and several release structures exist along the channel. Water from these release structures finds its way back into the Wimmera River through a variety of channels.

Normally diversions at Huddlestons Weir are only limited by the channel capacity. However the diversion is closed down when the storages are full and on some other occasions for example to exclude highly saline water during the first flow event after summer.

This system was radically simplified in the model. All the tributaries were treated as a single lateral inflow and all the release structures and their downstream channels were treated as a single structure with a short channel routing any released water straight into the river downstream of Huddlestons Weir. A special subroutine was written to determine the diversion and release discharges. It was assumed that the diversion was operated to allow for flows entering the channel from the various tributaries. Times when this diversion was inoperative were determined externally and supplied to the model via a binary timeseries.

Wetland sections, which are characterised by anabranching channels, were difficult to represent due to the lack of detailed data on the channel network in these areas. Accordingly, a single effective channel was used to model these sections rather than the actual channel network. This effective channel was obtained by estimating the number of anabranches in a particular section and multiplying the storage width and cross-sectional area of the measured or generated cross-section by the number of channels. Cross-sections were generated using distributions applicable to wetlands. This approach is justified by assuming that the anabranches are of similar lengths and thus have similar energy grade lines.

The extensive periods of low flow were also difficult to model. Small water depths cause problems with most finite difference solutions of the St Venant Equations (Cunge et al., 1980). MIKE 11 deals with this problem by introducing a slot into the bottom of the cross-section, thus artificially increasing the depth. If this slot dries out the model simply adds some more water to keep the solution running (DHI, 1992). Initially it was not possible to model adequately discharges less than 400 Ml/d in the Wimmera River. Such conditions are critical from a salinity perspective and exist for the majority of time in the Wimmera River.

This problem was overcome by introducing a series of weirs at high points in the river channel and adding artificial slots to the cross-sections. This created a series of backwaters during low flows and avoided the formation of shallow areas. During high flows the weirs became drowned. These weirs did not significantly affect the routing of hydrographs. This solution to the low flow problem is more difficult than this description implies because there are inevitably problems with oscillations at the weirs which are difficult overcome.

CALIBRATION AND TESTING.

Data available for calibration and testing of the model consisted of continuously monitored stage and discharge, the rating curves for each of five gauging stations along the Wimmera River and continuously monitored salinity at three stations along the Wimmera River.

Flow Resistance.

Manning's Equation was used to describe flow resistance with Manning's n determined by calibration. The three interrelated data sets available for calibration - timeseries of stage and discharge and rating curves - represent two separate groups of processes. Rating curves describe the relationship between discharge and stage which, given subcritical flow, results from downstream controls and flow resistance. On the other hand, the attenuation and timing of the discharge hydrograph depends on upstream storage processes. These are influenced by flow resistance, various controls and any off-channel storage. Interpreting stage timeseries is a little more subtle. The timing of fluctuations in stage depends on the timing of the discharge hydrograph and thus the upstream processes. However, for a given fluctuation in discharge, the fluctuation in stage depends on the downstream processes.

Because data for many inflows required infilling, errors in predicted discharge due to erroneous boundary conditions were expected. Any bias in the boundary conditions could potentially affect the calibrated value of n. This problem was avoided by calibrating n using the stage-discharge relationships. The predicted discharge was used to calculate an observed stage using the actual rating curve. The observed and predicted stage then relate to the same discharge and are comparable. This effectively removes the majority of any effect of erroneous discharge prediction on the calibration.
A value of $n = 0.06$ was chosen after three calibration simulations with $n = 0.05$, 0.06 and 0.07. The calibration period used was 15/6/89 to 10/10/89. These simulations indicated that values of n less than 0.06 were appropriate for 415206, 415239 (high flow gauge) and 415256 and n greater than 0.06 was appropriate for 415201 and 415200. Since the

	Gelnorchy 415201			Horsham 415200			Upstream Dimboola 415256			Lochiel 415246		
Year	Error (Ml/d)	Error (%)	Coeff. Effic.	Error (Ml/d)	Error (%)	Coeff. Effic.	Error (Ml/d)	Error (%)	Coeff. Effic.	Error (Ml/d)	Error (%)	Coeff. Effic.
1990	-2.76	-1.4	0.93	4.41	13.0	-0.45	23.67	78.3	-0.07	27.30	102.9	-0.09
1991	-22.20	-7.8	0.91	-36.03	-19.1	0.72	-54.95	-22.0	0.83	-67.39[a]	-19.4[a]	0.83[a]
1992	-13.65	-2.1	0.91	-196.6	-21.3	0.65	-30.84[b]	-4.8[b]	0.73[b]	-212.5	-20.4	0.72

Table 1: Mean annual errors in simulated discharge and coefficient of efficiency for the prediction of instantaneous flow for Wimmera River Gauging Stations. a - some low flows missing. b - some high flows missing.

stage discharge relationship predicted at a gauging station depends on the downstream cross-sections as well as the flow resistance, errors in the channel representation downstream of the gauge as well as incorrect n values will produce simulation errors. Calibration of n at individual gauges is therefore clouded by the use of stochastically generated cross-sections. For this reason and because no trend in n along the stream or with discharge was apparent a single value of n was used for all river reaches and for all discharges. It should also be remembered that, in this case, calibration data has been exclusively from gauging stations and that because of the way gauging station locations are chosen, these data are not necessarily representative of the entire stream.

Routing Performance.

With the model set up as described above and Manning's n set to 0.06, the wave celerity was considerably too fast. This indicates that the storage is being underestimated. This may be due to the calibrated flow resistance being too low. Simulations indicate a value of n = 0.10 produces a realistic wave celerity. Use of unrepresentative cross-sections could also be the cause though this is considered unlikely. It is more likely that off-stream storage, which has not been incorporated in the model, is important even at relatively small discharges. There are significant lengths of the stream where relict channels create backwater areas and the wetland sections of stream also tend to have a significant amount of low lying area that field observations show is inundated even at relatively low (1000 - 1500 Ml/d) discharges. The simplistic representation of wetland areas may also contribute to the observed error.

Given that little data relating to off-stream storage exists, a calibration approach was used to correct the erroneous wave celerity. This involved multiplying the storage width of all cross-sections by a constant factor, W. Simulations indicated that the coefficient of efficiency for the prediction of instantaneous discharge was insensitive to W. This insensitivity was partly due to the fact that many of the errors

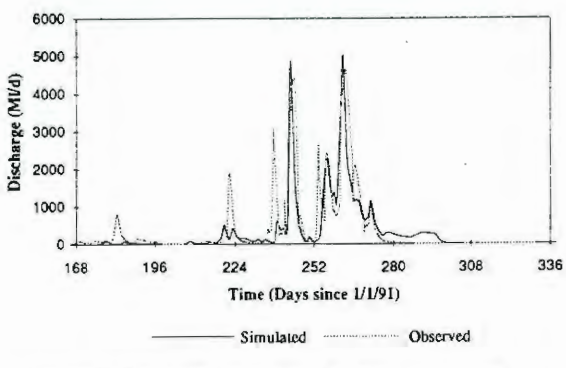

Figure 3: Simulated and observed discharge for the Wimmera River at Lochiel (415246).

observed were related to poor boundary conditions rather than the model parameterisation. A value of W=1.5, which is equivalent to an increase in storage width of 50% was chosen using the timing of the rising limb of the hydrograph as the main criterion. Although it is suspected that wetland areas were the major source of additional storage this increase in storage width was simply applied to the entire stream length. Wetland sections are distributed along most of the section of river modelled and since gauging stations are located at much greater separations than the wetland areas, no data for the relative wave celerity through wetlands and other sections of stream were available.

Once calibration was complete, a test simulation was conducted for the years 1990 - 1992. Figure 3 shows a representative hydrograph at Lochiel. Table 1 provides coefficients of efficiency (Aitken, 1973) and mean annual errors for the prediction of instantaneous flow at four gauging stations along the river for the three years of simulation.

The following general comments may be made. Discharges are well simulated at Glenorchy; however at other gauges the quality of simulations varies between events but is consistent between gauges. There is a tendency for peak and total discharges for events to be underestimated which indicates that a significant amount of error is associated with poor boundary conditions. Timing errors also occur; especially during large events. These are due to timing errors in inflows and incorrect representation of storage particularly during flood discharges.

Salinity Simulations.

No calibration of the solute transport model was necessary since simulations were insensitive to the dispersion coefficient which is the only tuning parameter. The dispersion coefficient was simply set using a deterministic relationship with flow velocity. The simulations presented here utilised monitored salinity data (8.3% of points infilled using a regression relationship with discharge) for the upstream salinity boundary condition.

Figure 4 shows simulated and observed salinities and the observed discharge at Horsham (415200) for the period 18/8/92 to 30/4/93. Unfortunately most of the data at monitoring station 415256 are missing for this period. During moderate to high flows the pattern of salinity fluctuations is correctly predicted although some systematic errors are observed. When the flow drops close to zero the salinity predictions become more erratic. This is likely to be due to the influence of local inflows of groundwater and storage effects in both the model and the river.

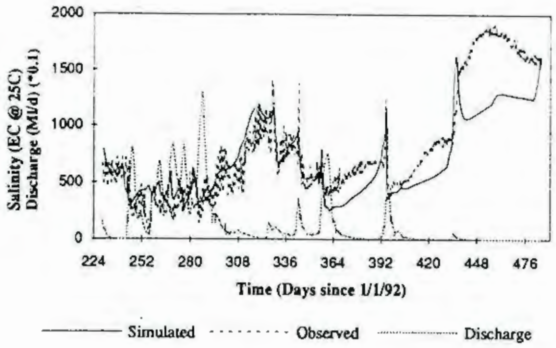

Figure 4: Simulated and observed salinity and observed discharge for the Wimmera River at Horsham (415200).

DISCUSSION.

During this study a number of issues relating to modelling technology and methodology have arisen. This discussion of these issues concentrates on the hydrodynamic modelling; however many of the comments apply equally to the solute transport modelling. The comments made apply to all physically based hydrodynamic models.

It is important to remember that we are modelling a relatively long section of stream with the aim of ultimately developing a planning tool. The model had to be capable of simulating individual events which required a time step of 3 hours or less during events. During the extended low flow periods the time step could have been significantly longer.

Data Limitations.

For most modelling studies there are only limited data available and limited time and resources for the collection of additional field data. In this study additional data collection concentrated on stratified pools and stream salinity monitoring which were both critical to the projects success. Data limitations in these and other areas have had a significant impact on this study.

The fact that many boundary conditions required infilling has limited the calibration and testing of the model. When significant uncertainty about boundary conditions exist it becomes difficult to separate simulation errors due to the representation of the stream and the representation of the boundary conditions. Even if the representation of the stream were completely correct we would expect simulation errors due to the boundary conditions. The conclusion of any model test is then inevitably along the lines of: the model performs as well as expected given the inadequate boundary conditions.

Calibration of the model also becomes difficult. If you are unsure of the channel representation downstream of a gauging station due to inadequate data (eg. infilled cross-sections) you can never be sure whether you are obtaining a representative n value or simply compensating for some error in channel representation. No model could overcome these data limitations.

Conceptualisation.

A purported benefit of models that solve the St Venant Equations directly is that they model the system using the physically correct equations. Indeed Cunge (1989) writes "the days of industrial use of 'simplified equation models' are over except for very special purposes". However when significant simplifications are required the purported benefit of using the St Venant Equations may not be realised.

In this study data limitations have resulted in wetland sections of the river being modelled with a conceptual channel and in significant simplifications when representing the Huddlestons Weir diversion. It is certainly not possible to make the claim that we have directly modelled the physics of the system.

Scale.

The model had to be capable of simulating solute transport for a major proportion of the main stem of the Wimmera River. It also had to be capable of simulating stratified pools which are characterised by a length scale of 100 m. A practical lower limit, given the planning objective, on the spacing of cross-sections in this application was ~1 km which resulted in a simulation time of 2 hours for 1 years flow and salinity using a Hewlett Packard 9000-720 workstation.

To build stratified pools into this model then requires either some form of grid or model nesting (increases computational time anyway) or the development of a conceptual model of saline pool processes and the averaging of saline water storage over a grid cell. A similar problem would exist if using a network model to describe wetlands since many of the anabranches are significantly shorter than 1 km. Thus there are practical limits on the range of scales that can be explicitly included in these (and other) models.

Scale is also important when testing models. Another purported advantage of physically based models is their ability to explicitly represent spatial variability. However it is often not possible to test this representation. Gauging stations are separated by an average of 45 km along the Wimmera River. Thus wave celerities obtained from the gauging data represent storage processes averaged at that scale. This is significantly greater than the scale at which the channel varies. It has not even been possible to test the accuracy of predicted wave celerities for the two major channel categories in the Wimmera River - wetlands and single channel sections.

A different perspective on scale and model testing is gained by considering calibration of n using rating curves (or stage data at gauges). These data represent the average effect of flow resistance and controls for a relatively short distance downstream of the gauging station. As a result the calibrated value of n only represents a small section of the river and stage predictions go untested in sections of stream remote from gauging stations.

Why Model?

We have highlighted a number of problems encountered in this study. One is then tempted to ask why model? Two primary reasons are that models provide a useful tool for critical analysis (Konikow and Bredehoeft, 1992) and a theoretical framework within which to carry out that analysis (Grayson et al., 1992). The act of developing a model can also provide new insight into the operation of a system (Konikow and Bredehoeft, 1992; Grayson et al., 1992). In this study the model has helped organise a large amount of disparate information related to the Wimmera River System. It has also provided insights into the operation of that system, particularly by indicating the importance of off-channel storage even for small flow events, although the limited data have prevented detailed testing of this hypothesis.

Model Alternatives.

We have indicated that there are a number of problems encountered when using sophisticated hydrodynamic models for modelling of water quality in large systems to provide long term information for planning purposes; especially when there are limited data available. We have also argued that there are benefits in developing models. Is there then an alternative to the use of sophisticated models in studies such as this?

A simpler model could be based on the Muskingum-Cunge routing scheme, which is equivalent to the diffusive wave approximation of the St Venant Equations (Cunge, 1969) and Fischer's (1972) lagrangian advection dispersion model which avoids the time step limitations of many eulerian finite difference models and does not introduce the numerical dispersion problems associated with many simpler transport algorithms. This model would be computationally less intensive, would simulate low flow conditions more easily and could be easily coded for this study.

General Models.

One problem that can be encountered when using generalised models is that there are features of the system that cannot be adequately modelled with the available algorithms. For example the diversion at Huddlestons Weir required additional subroutines to be written. The facility to add algorithms to generalised models is therefore a significant advantage. The opposite problem can also occur when there is an algorithm designed to automatically cope with a given situation; for example small depths. Such algorithms can limit the models applicability. On the other hand writing code to solve a specific problem has the advantage of flexibility, particularly when unusual situations are encountered.

Generalised models also have the potential for abuse. Probably the most important step in any modelling exercise is the conceptualisation of the physical system. This requires a proper understanding the processes likely to be operating in the system. Generalised models typically make a range of sophisticated simulation algorithms available in packages that are becoming increasingly user friendly. It then becomes possible for a person who is unfamiliar with computational hydraulics to obtain results based on incorrect or inadequate assumptions. This substitution of sophistication for understanding (Grayson and Nathan, 1993) is an issue that model developers, practitioners and educators must face as a matter of priority.

SUMMARY.

This paper has described the application of a physically based numerical model to the prediction of discharge and salinity in the Wimmera River. It was developed with the ultimate aim of providing a planning tool. The model is a large scale model in the sense that it applies to a major proportion of the length of the Wimmera River.

A number of issues relating to the use of such models for planning purposes are discussed. These include problems related to the lack of data which lead to difficulties in calibrating and testing models and to the conceptualisation of at least some system components. This tends to compromise the purported benefits of the models physical basis. Furthermore the scale of application of such models places a lower limit on the scale of features that can be described. It is also difficult to test the detailed or small scale predictions of such models due to large data requirements. An alternative model is proposed and modelling benefits and generalised models are discussed. It is suggested that model selection is a critical step in the modelling process and that simpler models do still have a place in hydraulics and hydrology.

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